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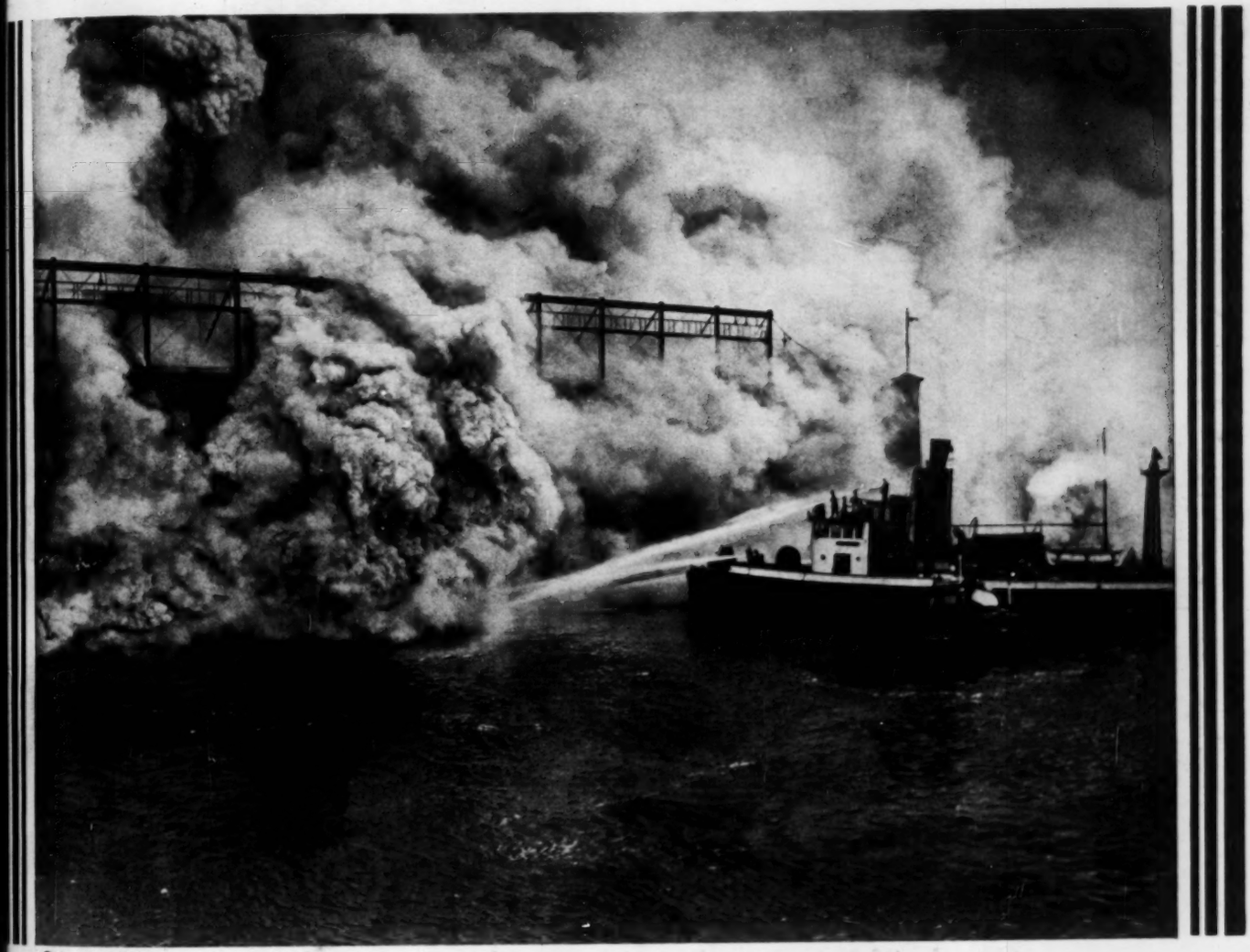

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CUNARD PIER 54, NEW YORK, N.Y., AT THE HEIGHT OF THE FIRE WHICH DESTROYED IT

Volume 3 — Number 10

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

OCTOBER 1933

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VOLUME 3

NUMBER 10

Richmond Launches Program of Bridge Construction

A Public Works Plan for Unemployment Relief in Virginia

By S. C. JEMIAN
ASSOCIATE MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS
FORMERLY DESIGNING ENGINEER, RICHMOND BRIDGE CORPORATION, RICHMOND, VA.

TOLL is to be charged for passage over the bridges now being built for the City of Richmond until the loans made for their construction by the Reconstruction Finance Corporation are repaid. The structures then will be the property of the city for the free use of its citizens. From the standpoint of design these structures have a number of unique features, among them the general use of rigid frames for the viaducts, and for the superstructure of the arches of the James River Bridge. Special and unusual treatment of the columns provides for movement due to expansion.

IN the present economic situation, the City of Richmond, like many other cities in the United States, has been impelled by the spirit of reconstruction to build roads and bridges in order to relieve unemployment and at the same time to pursue the ultimate aim of improving traffic arteries. With these two distinct purposes in mind the Richmond Bridge Corporation was organized to build several bridges. The entire project is being financed by the Reconstruction Finance Corporation. Tolls will be charged at all the bridges until the bonds of indebtedness have been retired, when the tolls will be abolished and the structures will become the property of the City of Richmond. Each of these bridges has several distinctive features of interest to structural engineers.

JAMES RIVER BRIDGE

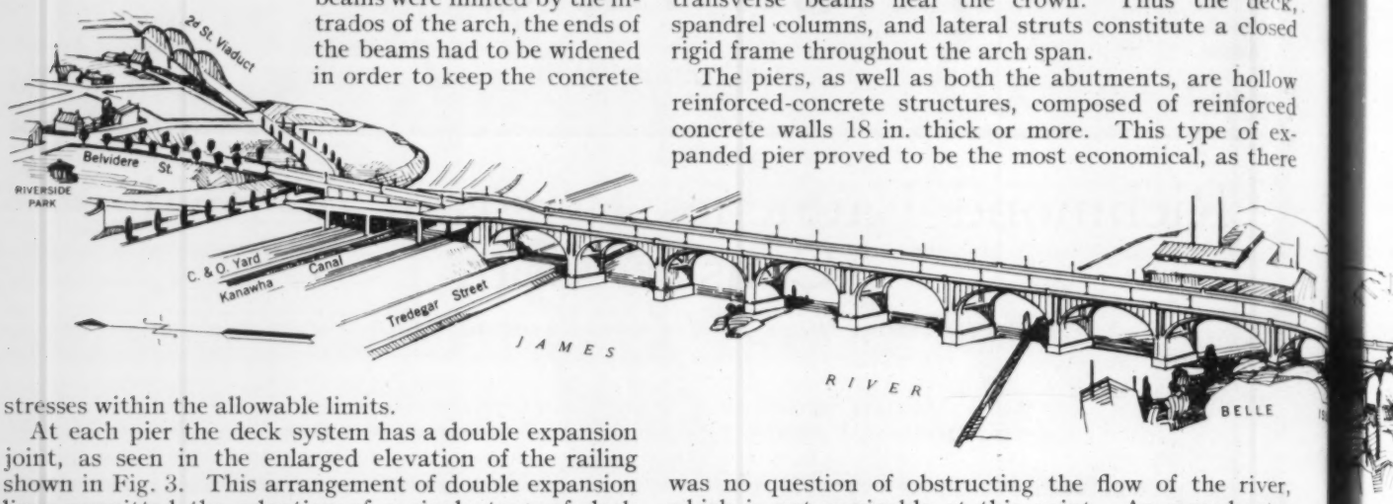
One of the most historic streams in America flows under the James River Bridge. Just east of this structure is the point where Captain John Smith and a few followers stopped in their exploration of the James River and set up a large cross on the shore. Belle Isle, at about the center of the bridge, is the site of one of the earliest iron foundries in this country. In Civil War days most of the armor plates for the first ironclad vessel, the *Merrimac*, were cast on this island.

The James River divides the City of Richmond in two sections and is spanned at several places by steel bridges, which are either off the main traffic artery or inadequate for heavy traffic. The James River Bridge crosses the main tracks of the Southern Railway, the James River, the Chesapeake and Ohio Railway main line and yard tracks, and the Kanawaha Canal, at a grade that connects the average heights of the north and south banks of the river. It will provide a direct connection for traffic through the City of Richmond on the relocated U.S. Route 1 and on U.S. Route 60. The bridge consists of 16 arch spans and several rigid-frame approach spans, aggregating a length of 3,847.67 ft end to end. A general idea of the dimensions and design of the bridge may be had from Figs. 1 and 3. The design of the cross section was primarily governed by the clearance requirements of the several railroads and other interests, thus limiting the length of the piers normal to the roadway to 30 ft. This limitation produced a cross section with long cantilevers to accommodate the 40-ft roadway and the two 4-ft sidewalks. Similarly, the lengths of the arch spans were governed by the clearance requirements of the private properties located on Belle Isle or otherwise crossed by the bridge right of way. The present arrangement proved satisfactory to the interested parties and was finally adopted. Thus the dimensions of the spans, instead of being governed by theoretical and economic principles, were governed by clearance requirements. The task was then to design the most economical section within given limitations. Fortunately the topography allowed considerable leeway in fixing the elevation of the roadway; so it was possible to adopt the most economical rise for a given span.

The bridge proper has a width of 48 ft. The roadway slab, 6 in. thick, is supported by 10 by 10-in. longitudinal beams, which have haunches 6 in. deep in order to keep the shearing stresses within the allowable limits. The concrete was stressed to its allowable limit except in certain parts where esthetic considerations necessitated an enlarged section. The transverse beams required a somewhat unusual design because it was necessary to conceal, as well as to support, the public utility conduits and the water and gas pipes, the weight of which total about 2,000 lb per foot of bridge. For this reason, and because of esthetic considerations, the transverse beams were lowered, as shown by the isometric elevation of the typical beam, *FB*, in Fig. 3. The deck load was then transmitted to transverse beams through 10 by 14-in. stub columns. Two curtain walls parallel to the bridge axis, and extending between the spandrel columns, conceal all the public utility equipment.

Though these transverse beams have somewhat the outline of a Vierendeel beam, the tremendous difference in stiffness of top and bottom chords did not warrant

such a design. They were analyzed as restrained beams where spandrel column stiffness justified the assumption; elsewhere they were analyzed as horizontal members of rigid frames composed of two spandrel columns and a transverse beam. Since the bottom elevations of the beams were limited by the intrados of the arch, the ends of the beams had to be widened in order to keep the concrete stresses within the allowable limits.



At each pier the deck system has a double expansion joint, as seen in the enlarged elevation of the railing shown in Fig. 3. This arrangement of double expansion lines permitted the adoption of a single type of deck framing for all the arch spans, regardless of the variation in the length of the pier spans. In order to keep the deck of all the arch spans of a constant length, the narrow strips of deck on top of the piers varied in width according to the width of the piers below; that is, the strip over an abutment was wider than that over a typical narrow pier. The narrow expansion decks were placed on separate rigid frames supported on the tops of the piers, there being a single rigid frame on each typical pier and a double rigid frame on each abutment pier.

The transverse beam reaction is transmitted to the arch through 20 by 24-in. spandrel columns, which have been designed as vertical members of rigid frames composed of these columns and the transverse beams.

Each span has two arch ribs 5 ft wide, 3 ft thick at the crown, and 4.25 ft thick at the springing line, with a clear span of 183.33 ft at the springing line. Both the intrados and the extrados are segments of circles. The arches were analyzed by the principle of the "Ellipse of Elasticity," described by A. C. Janni, M. Am. Soc. C.E., in Hool's *Reinforced Concrete Construction*, Vol. 3, and in TRANSACTIONS, Vols. 88 and 91, pages 1142 and 459, respectively. The design was based on the assumption that the arch ends are fully restrained, an assumption justified by the comparative stiffness of the arches and piers. A variation in temperature of 40 C was considered in addition to vertical loads, and the effect of shrinkage was eliminated by the introduction of five key sections, each about 7 ft long. The concrete of the key sections is to be poured not less than seven days after that of the arch segments. The arch reinforcement consists of top, bottom, and side bars, and ties of $^1/_2$-in. diameter. The pairs of arch ribs are laterally braced by means of eight struts and two transverse beams near the crown. Thus the deck, spandrel columns, and lateral struts constitute a closed rigid frame throughout the arch span.

The piers, as well as both the abutments, are hollow reinforced-concrete structures, composed of reinforced concrete walls 18 in. thick or more. This type of expanded pier proved to be the most economical, as there was no question of obstructing the flow of the river, which is not navigable at this point. As regards stability, these piers are very satisfactory on account of their wide bases and rigidity of section. A maximum moment of inertia is obtained for a given quantity of concrete. All the exterior walls are stiffened by vertical diaphragm walls 18 in. thick and by horizontal slabs about 15 ft on centers. Special care was taken to reinforce all corners efficiently in order to obtain rigid-frame action of the walls in all directions. At the typical narrow piers, the thickness of all the lateral walls, for the width of the arch ribs, was increased in order to obtain a better support for the vertical reactions of the arches. The adjacent skewbacks were made continuous and were given a well reinforced section 5 by 5.5 ft, thus realizing the perfect continuity of the arch rings.

SPECIAL TREATMENT OF ABUTMENT PIERS

The abutment piers were treated somewhat differently from the typical narrow piers. Since the rigidity of the arch supports was of prime importance on account of the unbalanced arch thrust during construction, a system of double walls was used, separated by a continuous cell extending to the foundation mat. These cells are to be filled with 1:4:8 concrete, thus providing a solid vertical support for the entire width of the pier. Here also, the continuity of adjacent arches is obtained through a horizontal section 5 by 5.33 ft between the skewbacks.

All piers located in water will have their cells filled with 1:4:8 concrete from the base mat up to high-water level in order to counteract the freezing action of the

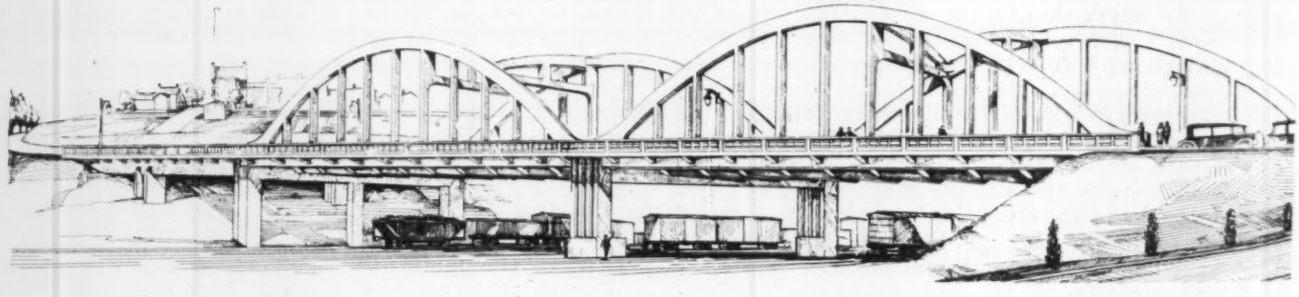


FIG. 2. PERSPECTIVE OF SECOND STREET VIADUCT OVER THE TRACKS OF THE CHESAPEAKE AND OHIO RAILWAY
A Skew Angle of 43 Deg Introduced Complications in These 147$^1/_2$-Ft Spans

water. Although all the foundations are on solid rock, the maximum pressure is but 5 tons per sq ft because of the large area of the piers on their foundations. No doweling of mats to base rock is provided for except at the abutment piers, where additional safety justified it.

SPECIAL DESIGN OF NORTH APPROACH

The north approach of the James River Bridge constituted one of the main problems on account of the strict requirements governing clearances over the railroad tracks, the Kanawaha Canal, and existing industrial facilities, and because of a street crossing under the north end of the approach. It would not have been difficult to bridge all these obstacles by continuous rigid frames if it had not been necessary to make special provisions for expansion. In order to solve the problem of expansion joints it was necessary to bridge the north approach with a series of single-span and double-span rigid frames. The single-span frames consist of a fixed column, a girder, and an expansion column, or a reversed L-shaped frame having one end of the girder ideally restrained and the other supported on an expansion column. The double-span rigid frames consist of a two-span girder having a middle fixed column and two end expansion columns. The expansion columns are in general 20 ft high. Though wide enough properly to carry the vertical loads assigned to them, they are very shallow and consequently will develop little resistance to temperature action. As the fixed columns have a much larger moment of inertia, changes in temperature will not appreciably affect them but will move the tops of the expansion columns back and forth. All the rigid frames were designed for a change of temperature of 40 C in addition to the vertical loads.

It may be of interest to explain the procedure applied by Mr. Janni to all problems of expansion. First the horizontal deflection, Δl, of the top of the girder between a fixed support and an expansion column is determined, as shown in Fig. 4, by the equation:

$$\Delta l = etl \qquad [1]$$

in which e = coefficient of expansion
t = temperature
l = length of span

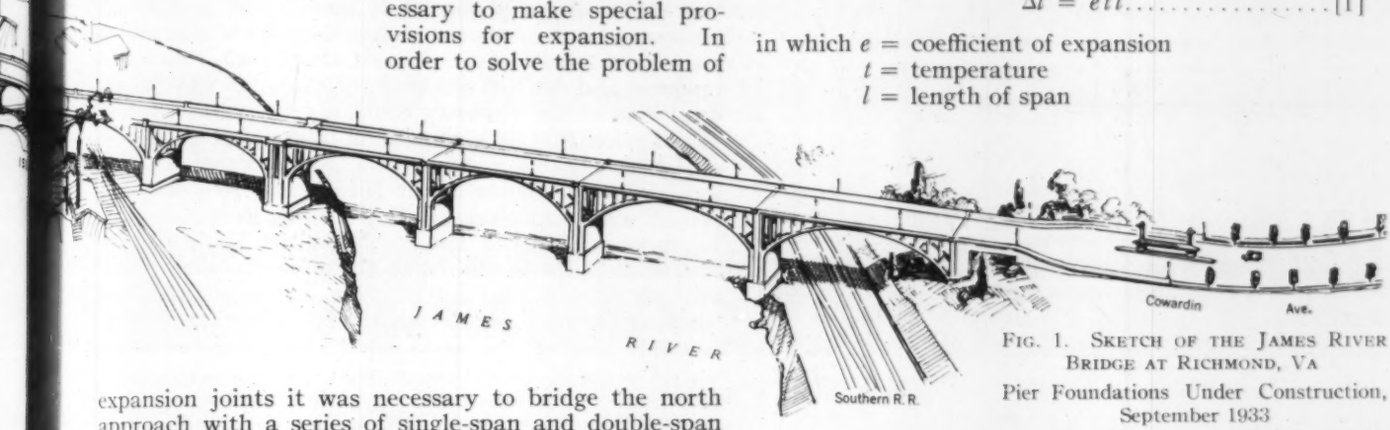


FIG. 1. SKETCH OF THE JAMES RIVER BRIDGE AT RICHMOND, VA
Pier Foundations Under Construction, September 1933

After determining Δl, the expansion column is treated as half of an ideally restrained beam having a span twice the height of the expansion column, and acting under a single load, P, at midspan. This similarity is justified when it is considered that the expansion column is rigidly attached to the girder. The tangent to the line of deformation of the expansion column, at the joint B, can be considered practically vertical, or in this case parallel to the tangent at the base, which is vertical, since the expansion column is restrained at the base. If the restrained beam shown in Fig. 4, having a span equal to twice the height of column, is loaded with a concentrated load at the center, the tangent to the line

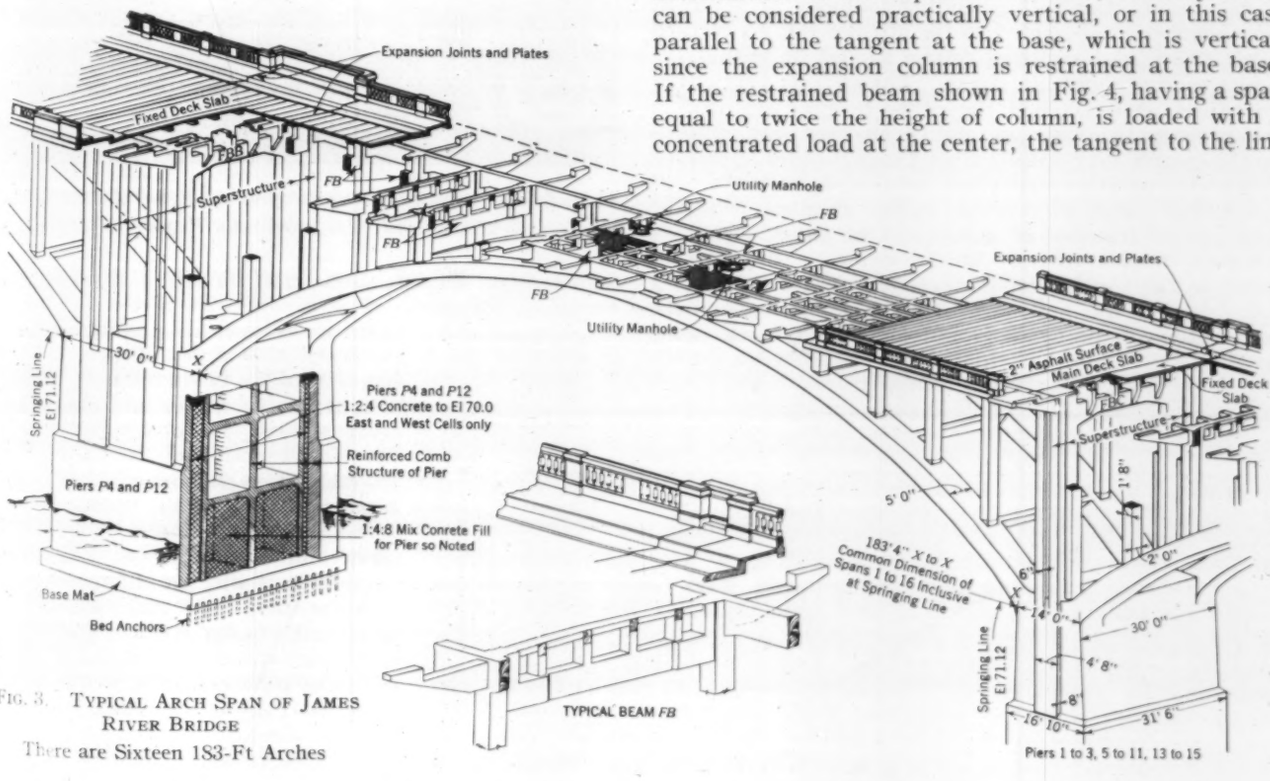


FIG. 3. TYPICAL ARCH SPAN OF JAMES RIVER BRIDGE
There are Sixteen 183-Ft Arches

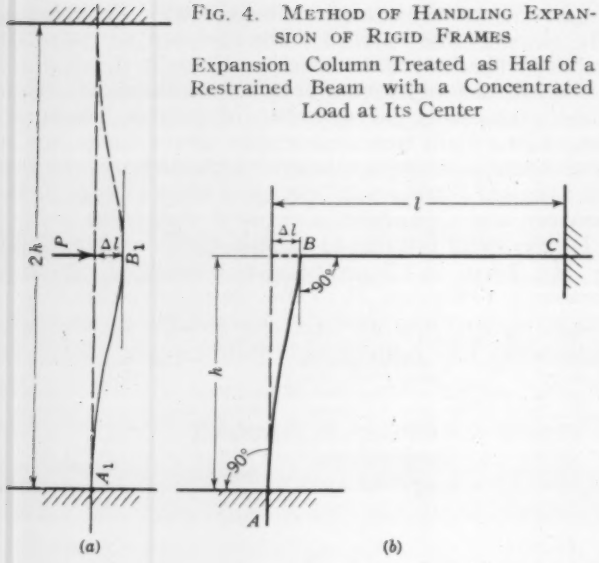


FIG. 4. METHOD OF HANDLING EXPANSION OF RIGID FRAMES
Expansion Column Treated as Half of a Restrained Beam with a Concentrated Load at Its Center

of deformation at the center will be parallel to the tangents at the ends of this beam. Therefore the analysis for both cases is identical, and the column is treated as half of a restrained beam, in which, by elementary statics:

$$\Delta l = \frac{1}{192} \times \frac{P\ (2h)^3}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots [2]$$

Consequently

$$P = \frac{\Delta l \times 192 \times E \times I}{(2h)^3} \quad \ldots\ldots\ldots\ldots [3]$$

and,

$$M_t = \pm \frac{P\ (2h)}{8} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots [4]$$

in which M_t equals the bending moment at A or B. The effect of M_t is added to the stresses due to vertical load. The total stress of course should be within the limits of the allowable stress. Throughout this entire structure, as well as in the other bridges included in the project, the same method of analysis was applied. In the case of the Second Street Viaduct other influences besides temperature were taken into consideration.

Another point of interest, rather original, was the question of freedom of movement of these expansion columns, buried entirely in the ground. In order to preserve the freedom of movement of these columns, a concrete shell was built around each, extending from the base of the column to the ground level. This shell, being fixed in the ground, provides ample space inside for the column to move back and forth. The arrangement is illustrated in Fig. 5, which also shows a typical double-span frame with fixed and end expansion columns.

The City of Richmond desired to create another artery connection on the north end of the James River Bridge, and Second Street was found to be the most logical street for this purpose. Since at the south it comes to a dead end at the Chesapeake and Ohio Railway tracks, an overpass bridge was imperative. Its location with respect to the James River Bridge may be seen in Fig. 1. The clearance for railroad traffic and the unlimited height above the roadway were the governing factors for the design. A bowstring arch proved to be the most logical type and was adopted.

The bridge has two arch spans about 150 ft long and two girder spans about 60 ft long. The arches are parabolic, have a constant section 2 by 3.5 ft high and a theoretical span of 147.5 ft, and carry a 40-ft concrete roadway and one 4-ft sidewalk. Despite all efforts, the dead load of the roadway could not be reduced below 190 lb per sq ft of deck on account of the 40-ft clear span of the floor beams, which were 12.8 ft on centers. This weight for the James River Bridge and its north approach was kept below 150 lb per sq ft, including 15 lb for paving. Thus in the Second Street Viaduct every inch of the arch is utilized to the allowable limit. The analysis showed that the pressure line was within the limits of the core and deviated very little from the geometrical axis of the arch, which fact warranted the use of a constant section of arch, with constant reinforcement. The horizontal tie girders have a section 2 ft wide and 2.5 ft deep. The arches were analyzed as restrained and supported on elastic columns, and the columns considered as rigidly attached to the arch ends.

Here too the question of expansion was of prime importance. The same solution was used as on the James River Bridge, each span having a fixed and an expansion support. For the Second Street Bridge, a single expansion column was found to be inadequate on account of the heavy vertical reaction as well as the bending of the column due to elongation of the tie girder and changes in temperature. To solve this problem, Mr. Janni conceived the idea of introducing two shallow columns at the expansion ends of the spans. These columns, acting simultaneously, will have ample capacity to carry the vertical load without the tremendous increase in the resistance to bending that would occur if the column sections were united. This arrangement of double columns can be seen in Fig. 2.

Noteworthy is the arrangement of the lateral bracing of the arches. Since the bridge is on a skew of 43 deg

FIRST STREET VIADUCT UNDER CONSTRUCTION, SEPTEMBER 6, 1933
The Narrow Piers Provide for Expansion

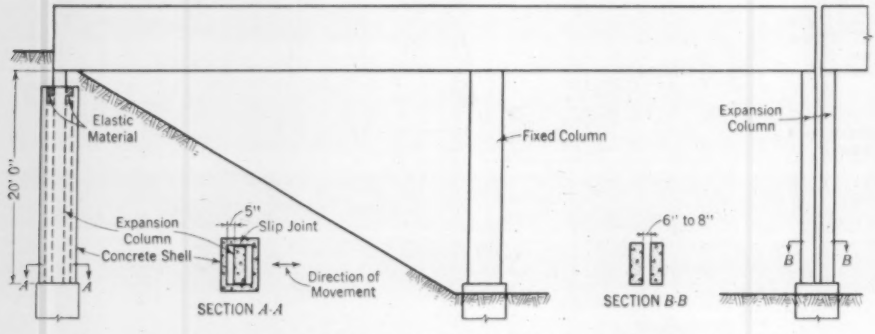


FIG. 5. ARRANGEMENT OF THE DOUBLE-SPAN, RIGID-FRAME APPROACH
End Expansion Columns Are in Separate Wells to Provide Free Movement

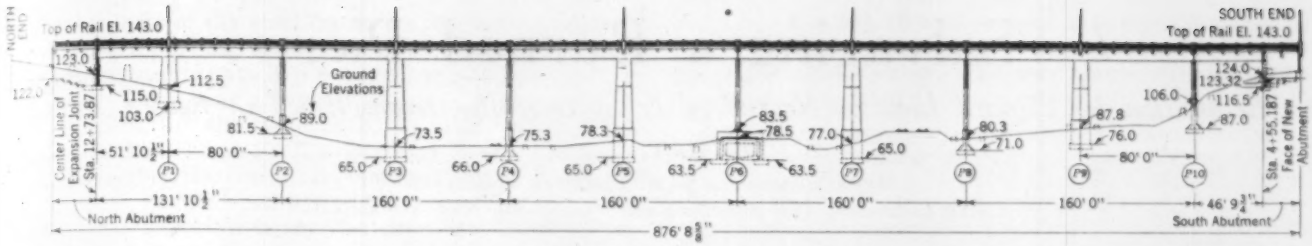


FIG. 6. ELEVATION OF THE FIRST STREET VIADUCT

5 min, there were only two points at which the two arches of the same span could be adequately tied together laterally without restricting the vertical clearance for traffic. These two points being at different elevations, the laterals were formed like semi-parabolas with horizontal bottom chords supported on the arch at one end and on the hangers at the other end. This detail is shown in Fig. 2.

Another point of interest is the formation of the floor beams. Their clear span of 40 ft required a minimum depth of 4 ft 6 in. at the center. This depth was decreased to 2 ft 6 in. at the ends in order to reduce the depth of the longitudinal tie girders; otherwise these girders would give the impression of being the main members of the span, thus eliminating the esthetic function of the arches. The decrease in the depth of these cross-beam ends was made possible by a gradual widening of the beam ends from 20 to 28 in. In this way the unit shear was kept within the maximum allowable limit.

The First and Fifth Street bridges were built to replace existing steel viaducts which are considered inadequate for present-day traffic intensity. They are

SECOND STREET VIADUCT PARTLY COMPLETED, SEPTEMBER 6, 1933
Expansion Column at Right End of Completed Arch

identical except in total length; the First Street Viaduct has a total length of 876.72 ft, and the Fifth Street Viaduct one of 1,185 ft. The bridges consist of a number of double-span rigid frames having a central tower support and two end expansion columns 14 in. by 3 ft in cross section and 20 ft in height (Fig. 6). The two adjacent expansion columns are supported by a single pedestal column, which extends to the foundation mat with very little increase in cross section. The central towers are actually U-shaped walls 14 in. thick, with 14 by 18-in. lateral braces spaced vertically 15 ft on centers. The isometric drawing in Fig. 7 shows details of design of both viaducts. The double-span rigid frames were analyzed as such for full vertical loading and a variation in temperature of 40 C.

In addition to the structures that have been mentioned, the project includes the widening and strengthening of the Marshall Street Viaduct, a steel structure over 2,456 ft long from face to face of abutments. The roadway will be widened 3 ft and the two sidewalks 6 in. each. In order that this increase of 4 ft in deck width may not increase the loads on the steel towers and foundations the existing concrete sidewalks will be replaced by 3-in. plank sidewalks.

This viaduct, built about twenty years ago, was found to be altogether inadequate for modern traffic. The main trusses had been designed as simple trusses, supported on steel framed towers, but actually trusses and towers were rigidly framed into one another. Consequently, the truss members not being proportioned in accordance with the actual stresses, were badly deformed and the rivets at the truss supports were either loose or broken, as revealed by a careful field inspection. Thus the entire structure had to be redesigned as a continuous rigid frame and the members of all the trusses and towers reinforced accordingly. The new design showed plainly the causes of all deformations of, or breaks in, the members and connections.

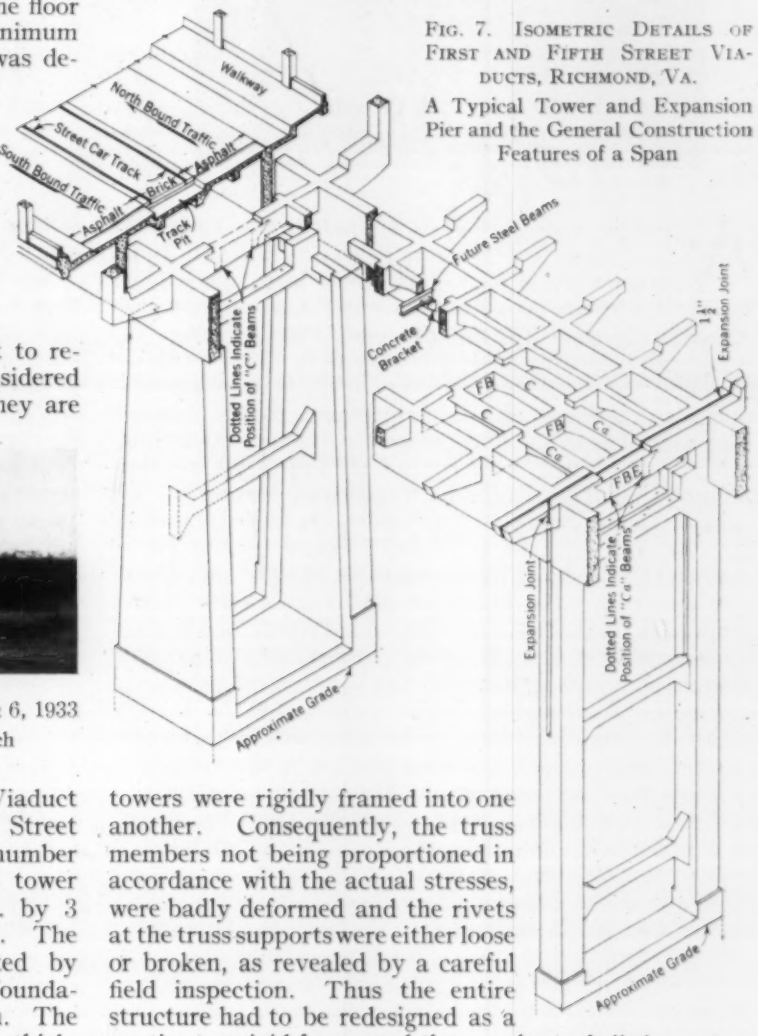


FIG. 7. ISOMETRIC DETAILS OF FIRST AND FIFTH STREET VIADUCTS, RICHMOND, VA.
A Typical Tower and Expansion Pier and the General Construction Features of a Span

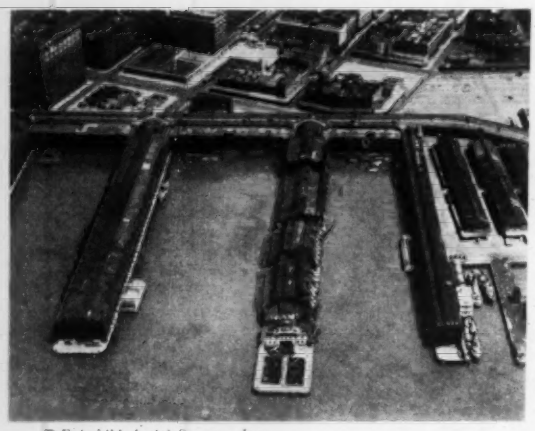
State Senator John J. Wicker, Jr., originated the idea of founding the Richmond Bridge Corporation, of which he is the president. The engineers are Allen J. Saville, Inc., and Lee Smith and Van Dervoort, Inc., both of Richmond. The consulting engineer is A. C. Janni, M. Am. Soc. C.E., of New York, for whom I was in charge of the design.

Salvaging a Burned Pier

Restoration for Cunard Line in New York Is Successfully Accomplished in Record Time

By R. T. Betts

Member American Society of Civil Engineers
Chief Engineer, The Robbins-Ripley Company, New York, N.Y.



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ON May 9, 1932, Pier 54, one of the large structures of the well known Chelsea Docks in New York City, lay in ruins after a devastating fire. A view of the fire at its height appears on the cover of this issue, and a view of the ruined dock from the air may be seen at the left. Just a little more than eight months later the entire pier was restored, in better condition than ever, and back in commercial use. Such a noteworthy feat was only accomplished by careful planning and hearty cooperation. As Mr. Betts explains, the various contracts were made interdependent and dovetailed, eliminating delays. The result constitutes what some water-front engineers consider to be an epic in dock construction. This account has been condensed from a much longer paper, filed through Mr. Betts's courtesy in the Engineering Societies Library for the information of all interested engineers.

AS part of what is known as the Chelsea Improvement along the Hudson River, owned and built by the City of New York, three large piers, Nos. 53, 54, and 56, are leased to the Cunard Line for its trans-Atlantic service. The middle one, Pier 54 (Fig. 1), was totally destroyed by fire May 6–9, 1932, seriously crippling the steamship company. Its prompt restoration was therefore of vital importance to the Cunard Line as well as to the underwriters. Some of the technical aspects of this problem and its successful solution suggest a number of valuable engineering lessons.

As originally built 25 years before, the substructure of Pier 54 and adjacent bulkhead platforms consisted of wooden pile bents on 10-ft centers for the pier and about 6-ft centers for the bulkhead platforms. These bents supported wooden caps, wooden foundations for the shed columns, wooden longitudinal stringers, and wooden plank decking, together with the usual wooden bracing on pile bents and wooden fender system. On the wooden plank decking was placed 6 in. of reinforced concrete covered with a wearing surface of 2 in. of sheet asphalt.

There were no fire walls or checks in the substructure and no fire manholes except one at the entrance, in the deck of Pier 54; but there was a reinforced concrete fire wall in the bulkhead platform 150 ft north of Pier 54, on the mid-line between Piers 54 and 56, and another similar wall 125 ft south of Pier 54 (Fig. 1). There were no fire curtains or draft stops in the superstructure, which had two floors, with a steel framework.

According to the official report of the examiners of the New York Board of Fire Underwriters dated May 26, 1932, the fire, which was confined almost entirely to the substructure, started about 7.00 a.m. on Friday, May 6, 1932, in the substructure of the bulkhead platform north of Pier 54. From there it spread north until stopped at the concrete fire wall between Piers 54 and 56 and south until halted about 80 ft south of Pier 54; also outshore, involving the substructure of the entire pier. The skeleton construction of the extension at the outshore end of the pier was only slightly damaged, principally at its connection with the original pier. The fire was finally extinguished early in the forenoon of Monday, May 9.

The burning away of the substructure, which of course included the wooden grillage footings for the columns of the steel frame, brought about the collapse of the entire superstructure outshore of Bent 34, leaving the inshore, or 175 ft of the main pier shed and the adjacent bulkhead sheds standing in a more or less damaged condition. Outshore of Bent 34 the structure collapsed in four distinct sections, as shown in the aerial photograph.

Inshore of Bent 34 the superstructure had settled in variable amounts from about 1 ft 3 in. at the junction bent with the bulkhead shed to about 5 ft at Bent 34. These so-called "standing portions" were prevented

Table I. Details of Twelve Contracts Awarded

Contract Number	Kind of Work	Contractor	Begun (1932)	Finished (1932)	Amount
1	Emergency shoring, inshore section	George W. Rogers Company	May 12	May 23	$ 10,895.00
2	Demolition and removal of superstructure	John E. Smith Wrecking Corporation	May 20	June 29	72,152.34
3	Demolition of pier inshore of Bent 34 and north bulkhead	George W. Rogers Company	May 26	June 25	12,691.26
4	Additional cribbing and shoring in bulkhead	George W. Rogers Company	June 2	June 11	1,950.00
5	Removal, repair, and replacement of baggage conveyor	Charles H. Newman	June 14	During general contract	2,550.00
6	Demolition in bulkhead area north of Pier 54	John E. Smith Wrecking Corporation	June 15	June 24	2 080.00
7	Jacking up pier shed and bulkhead sheds	John E. Smith Wrecking Corporation	June 16	July 18	22,517.37
8	Substructure reconstruction inshore of Bent 34 and north bulkhead	Frederick Snare Corporation	July 18	Sept. 25	61,053.48
9	Substructure reconstruction outshore of Bent 34 and south bulkhead	Stillman-Delehanty-Ferris Company	Aug. 17	Oct. 19	193,967.25
10	Structural steel frame of superstructure	McClintic-Marshall Corporation	Sept. 1	Nov. 3	Included in Fuller contract
11	Superstructure pier-shed doors	J. Edward Ogden Company	Erection Nov. 14	Erection Jan. 6, '33	
12	Completion of reconstruction of superstructures	George A. Fuller Company	Sept. 26	Jan. 18, '33	420,274.61
				Grand total.....	$800,131.31

from collapsing by the strength and integrity of the adjacent parts of the steel frame and by partial support from wedged portions of burned timbers and other debris. The superstructure itself was practically undamaged by the fire. That same afternoon, May 9, I was called in consultation by the Underwriters Adjustment Committee, and on May 10 The Robbins-Ripley Company was retained as the consulting engineers for all necessary conferences, field inspections, plans, specifications, contracts, and supervision for the demolition and reconstruction of Pier 54 and adjacent bulkheads. About a month later this relationship was also entered into with The Cunard Steamship Company, Ltd.

All the following operations of shoring, demolition, and replacement had to be carefully correlated in order to prevent delays and thus minimize losses through inability to use the pier. The remarkably close schedule maintained in the sequence of dependent operations is indicated in Table I.

An examination made on May 10 indicated that the standing portions could be salvaged if immediate steps were taken to prevent further settlement. Meanwhile, changes were frequently occurring in the condition of the collapsed structure—pieces of debris were becoming detached and slipping into the water; the steel frame was changing in grade and alignment; the boilers and equipment were slipping and changing position; and sway rods were breaking. Although the sum total of all these movements was not large, it, together with the accompanying snapping and creaking, was sufficient to cause some nervousness and apprehension on the part of both workmen and engineers.

Time was therefore of the utmost importance. Hence, on May 12 a contract was awarded for cribbing and shoring six selected columns and girders in the north bulkhead shed and the four outshore center columns and three selected bents, Nos. 38, 40, and 41 of the standing portion of the main pier shed. Later five additional columns in the bulkhead area were cribbed and shored. The burned piles were sawed off at about mean tide and capped with 12 by 12-in. timbers to support the cribbing, blocking, shores, and wedges. Transverse and longitudinal plank bracing was also placed. Because of the tangled and wedged condition of the debris this work was extremely hazardous and had to be carried on with skill and caution. While the securing operations were in progress, specifications had been prepared for demolition and salvage. This work was divided into two parts, that outshore and inshore of Bent 34.

The most difficult part of this work was the removal of parts of the substructure, the boilers, and other machines in the old power-plant area. Because of the interference of structural members and temporary shores, a considerable part of the second-floor reinforced-concrete slab was removed and also a considerable section of the roof and shed framing. Through this makeshift hatchway the four boilers were then hoisted vertically over the top of the roof. The three turbo-generators were under water and had sunk in the river mud until they were almost inaccessible, but with the aid of a diver slings were attached and these machines were eventually raised and hoisted through and over the roof.

In the superstructure there were 33 complete shed bents to be removed, totaling 2,580 tons, which included the side columns and lower-story center columns, girders, trusses, floor beams, door-head girders, struts, cargo columns, side doors, end framing (including ornamental pilasters and electric sign), and the second-floor reinforced-concrete slab. In order to effect the salvage of those members that were to be used again, amounting to 683 tons, or about 26 per cent, and representing a net saving of $18,980, it was obvious that certain members would have to be sacrificed. Throughout the demolition and reconstruction, time was always of paramount importance; so that those members which were easy of

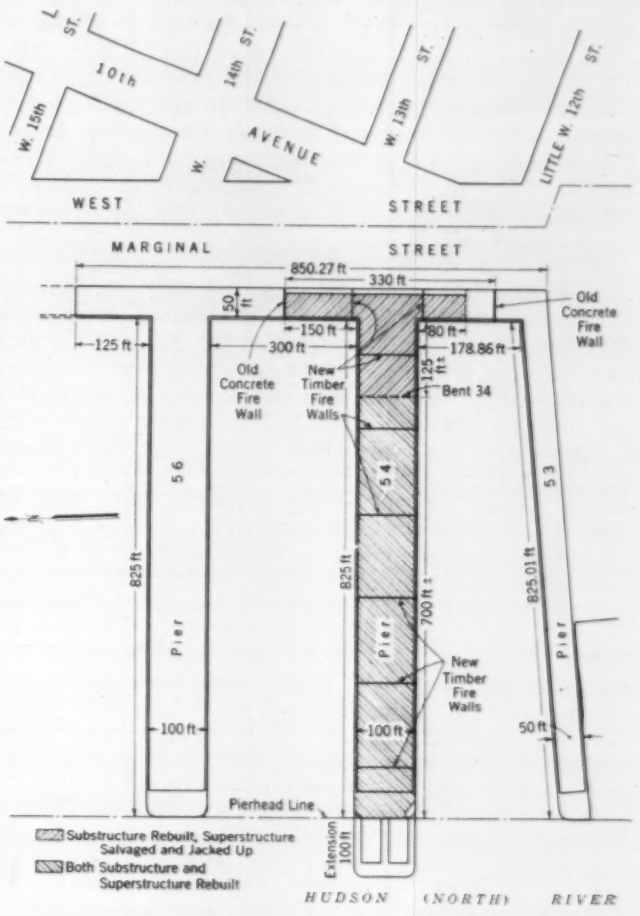


FIG. 1. LOCATION OF PIER 54, HUDSON RIVER, NEW YORK, N.Y. The 100-Ft Extension Was Only Slightly Damaged

access and would perhaps involve a longer time in the mill and shop to replace were selected for salvage. No attempt was made to recover many other usable pieces because their recovery would have materially slowed up the work. The second-floor slab was cut into squares by hand and lifted bodily to the debris scow.

Following the removal of this debris the slips on either side were swept by means of a steel cable sunk to the proper elevation and dragged by derrick boat along the river bottom. This method was unsatisfactory, as the cable would ride over obstacles without definitely indicating their position or projection above the bottom. An angle iron bar about 40 ft in length was substituted for the steel cable, and records were made of obstructions as noted. These were subsequently removed by means of divers and derrick.

While this work of demolition and clearing the site was in progress, specifications were prepared and a contract let to jack the standing section back into position. This was rather a complicated job because of the shores and cribbing that had been placed on the sawed-off burned piles to secure that part of the superstructure against further damage. Drawings were furnished showing the position both horizontally and vertically of all piles and timber work and also profiles giving the levels on the second floor of the structure and

the established grades to which this floor would have to be brought. Calculated column dead loads were also furnished the contractor for his guidance in preparing an estimate on the work to be done.

As this work proceeded, the travel of the jacks was taken up by blocking of yellow pine lumber and by oak wedges so as to maintain a tight contact at all times. Steel cables 1 in. in diameter were placed diagonally across the pier-shed crossing at the center to maintain the side columns in a vertical position, and similar longitudinal cables were placed to resist the outshore list of the structure. All cables were provided with turnbuckles. During the time the jacking was in progress the cables were somewhat slackened, but at all other times they were kept taut.

Observations from the second floor were taken continually during all the jacking operations. In addition to bringing the structure to the established grade along the edges, it was necessary to warp the crown of the pier shed into the level floor of the bulkhead near the main waiting room. All this was successfully accomplished and the structure was brought back to these grades with a variation of not more than $^1/_8$ in. The street-front concrete wall, which had been under close observation during the jacking operations, gradually came back to its original position as the steel frame righted. When the jacking was completed the wall was plumb.

Assuming that all the important members of the steel frame of the pier shed from Bent 34 inshore to Bent 40, inclusive, could have been salvaged, there was an estimated saving of $25,000 by retaining this standing section of the pier shed. Had it been necessary to demolish and rebuild the damaged north bulkhead superstructure, including about 70 ft of the ornamental reinforced-concrete street-front wall, there would have been an additional expense of about $35,000.

In the reconstruction of the pier substructure inshore of Bent 34 and the north and south bulkheads, all longitudinal timbers except the fender system were omitted. For the original deck system consisting of stringers, planking, and concrete were substituted a 10-in. reinforced concrete slab for the main pier and an 8-in. one for the bulkheads, protected by a 2-in. asphaltic surface. Concrete column pedestals were provided so that in the event of any future fire the shed columns would rest on fireproof foundations.

A careful inspection of the burned piles indicated that the full size of the pile and sound timber would always be assured at 3 ft 6 in. above mean low water, or about 1 ft below mean high water; so it was decided to rebuild the structure on the original piles. The pile cut-off for all this work was fixed at 2 ft above mean low water. The City's Department of Docks permitted the use of untreated lumber up to a plane of 3 ft above mean low water. Above that plane all lumber, except for the fender system along the sides of the pier, was to be creosoted.

In the final design of the substructure, three principal circumstances had a controlling influence: speed of completion; interference of jacks, shores, blocking, wedges, and stay cables; and the necessity for arranging the work so that the columns could be released and secured to the new concrete pedestals. Column footings were built in staggered lines starting with the side column in one bent, the center column in the next bent, and the other side column in a third bent. Thus a minimum number of jacks per bent were released in the initial stages and one column in each bent was secured prior to subsequent operations. The contract for the reconstruction of the substructure inshore of Bent 34 and the north bulkhead was awarded the same day that the jacking operations were completed.

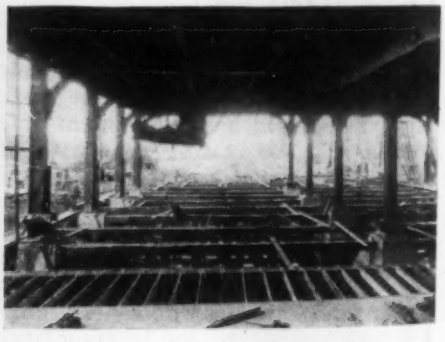
Creosoted wooden fire stops were constructed in the north bulkhead about 15 ft north of the north side of Pier 54 and in the main pier between Bents 37 and 38. These fire stops were built with a central core of solid creosoted 12 by 12-in. lumber between the pile caps and the under side of the concrete deck and extended the entire width of the pier. On each side of this central core two layers of 2-in. creosoted tongued and grooved sheathing were placed, extending from a bearing on the low-water horizontal braces to the under side of the concrete deck. The under layer was laid diagonally and the outer layer vertically, thus making the fire stops 20 in. thick. As no piles were broken in this area no new piles were driven.

Outshore of Bent 34 a generally similar arrangement was adopted, except that some details were heavier, as shown in Fig. 2. For example, $1^1/_2$ by 20-in. oak dowels were added to connect the posts to their piles. In this section many new piles had to be driven, to take the place of those broken or missing and they extended up to frame into the deck cap. The same was true of the new piles added at each end of the pile rows. Outshore of Bent 34 both new and old piles for the column footings were cut off 2 ft above mean low water. All the other old pier piles were cut off at a height of 3 ft above mean low water.

Creosoted wooden fire stops of the same type used on the inshore section were placed at intervals of 140 ft along the pier, as shown in Fig. 1. Reinforced concrete fire walls were considered but because of the flexibility of the structure, the soft bottom, and the deep water (40 ft at mean low water along the sides of the

Superstructure Removed, June 17, 1932

Bents and Footings Under Construction, September 19, 1932



Inshore Section Jacked up to New Foundations, September 8, 1932

STEPS IN RECONSTRUCTION OF PIER 54, NORTH RIVER, NEW YORK

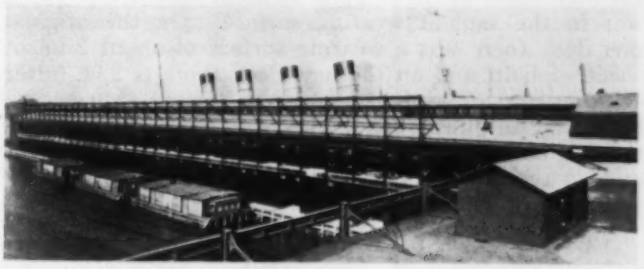
pier, they would be likely to crack when the pier was in use. A further consideration was the damage that would probably result from the freezing and thawing of tidal waters and from floating ice and debris. It was therefore decided to adhere to the wooden construction, which, while conforming in a general way to minor movements under operating conditions, would provide considerable lateral stiffness in the substructure.

All the concrete pedestals as well as the concrete floor slabs outshore of Bent 34 were constructed with rapid-hardening cement. Standard portland cement was used for the floors inshore of Bent 34. This use of rapid-hardening cement accelerated the work and actually saved 8 days of time.

After the site had been cleared and all the piles that appeared to be broken had been removed, tests were made of all the piles in the shed foundations and of approximately 90 per cent of the remaining vertical piles supporting the pier to see whether or not any unsuitable piles had been overlooked by the demolition contractor. Each pile tested was subjected to a vertical pull by means of a tackle at the end of a derrick boom, and the resulting strain was calculated from the displacement of the derrick scow. The average strain was about 20 tons.

Although quite familiar with this section of the North River and reasonably sure of the length of piles required, we decided to attempt to remove some of the original piles to check up on their lengths. A record of the original pier construction, which showed the position of unlagged, half-lagged, and fully lagged piles, was furnished by the Department of Docks, and from this four piles were selected, all on the north side of the pier. A two-block five-part purchase, using a $^3/_4$-in. steel cable and a 2-in. bull chain, was attached to lashing in pile-driver leads and was used for the four piles. The first pile was an unlagged pile 73 ft long in the work, with a 7-in. point, a 14-in. butt at the cut-off, and a penetration of about 34 ft. It was pulled out at the end of about 35 min. after an actual strain of about 31$^1/_2$ tons. The second pile was half-lagged, and broke off about 8 ft below the under side of the cross cap after a strain of about 60 tons at the end of about 20 min. Before the third attempt, also on a half-lagged pile, an effort was made to break the frictional resistance or "set" with 52 blows of a 4,200-lb hammer falling 12 ft, which drove the pile in about 8 in. The strain was then applied for 2 hr 30 min at a maximum of about 63 tons. As there was no evidence that the pile was yielding, the attempt was abandoned. The fourth pile was also half-lagged. First it was subjected to a pulling strain for about 15 min and then to 27 blows of a 4,200-lb hammer falling 12 ft, which drove it in about 11 in. Then the strain was again applied and the scow was pulled down as far as was considered safe. At the maximum strain the lashing in the pile-driver leads, consisting of 7 turns of 2-in. new manila rope, broke and the attempt was abandoned.

These results were not unexpected. They justified the decision to build up from burned piles, thus obtaining the value of the long "set." All new brace piles are 75 to 79 ft long and half-lagged. All the new vertical piles in a zone 21 ft wide along each side of the pier are 80 to 84 ft long, and in the central 58-ft zone they are 75 to 79 ft long. The north corner was entirely rebuilt with piles 80 to 84 ft long.



RECONSTRUCTED PIER 54 READY FOR SERVICE ON JANUARY 5, 1933
Mauretania Docked on North Side of Pier

In the original pier, from the bulkhead wall at the street front to the outshore double row, there were 2,977 vertical piles. To replace broken and missing piles and provide one new pile at each end of each single pile row, 740 vertical piles were required, all outshore of Bent 34. In the original pier there were 90 brace piles, of which 38 were replaced. In addition, 64 new piles were driven, one at each end of each single row. All new piles were cleaned of bark above mean high water and liberally swabbed on the top and sides with creosote oil.

For the reconstruction of the entire substructure, including the north and south bulkheads, approximately 1,230,000 fbm of lumber was used, of which 785,000 fbm was creosoted. Of the untreated lumber, about 130,000 fbm was used in the lagging of the pier piles.

Before and while the substructure was being reconstructed, plans and specifications for the structural steel frame of the superstructure were prepared; so that erection would not be delayed. Under this contract all the new structural steel outshore of Bent 34 and that in the north bulkhead was furnished and erected in place at a certain price per pound, as was all the salvaged structural steel.

DEMOLITION OF THE PIER IN PROGRESS, MAY 27, 1932
Some of the Roof Trusses Were Salvaged for New Construction

Three tractor cranes were used for the steelwork, one for sorting and distribution, and the other two for actual erection. Each crane was of 8-ton capacity at a 15-ft radius and had a 70-ft boom, lengthened to about 76 ft by means of a jib. The use of these tractor cranes was permitted under the following conditions: the reinforced concrete deck was to be not less than six days old before it was subjected to any load, and the skid timbers were to be 14 in. deep and long enough to distribute the load over three pile bents. The work was completed without injury to the concrete deck or any part of the structure.

The last contract for restoring and reconstructing the remaining parts of the superstructure covered extensive work, including roofs, monitors, ornamental steel and

cast iron, concrete floor, asphalt wearing surfaces, office quarters, stairways, and fire curtains. In addition it included the repair and restoration of standing parts at the bulkhead end to their former state, and the installation of fire lines, plumbing, electricity and power service, elevators, and final cleaning and painting.

In general the materials and workmanship were identical with those in the original structure. One exception was in the asphalt wearing surface. On the original pier deck there was a wearing surface of about 2 in. of sheet asphalt, and on the new deck there is 2 in. (after compression) of asphaltic concrete surface course consisting of bitumen, sand, and trap-rock tailings. On the original second floor there was $^1/_2$ in. of rock asphalt, and on the new floor there is about $^3/_4$ in. (after compression) of asphalt mastic.

AN ECONOMICAL JOB

For all the demolition and reconstruction work 12 contracts were prepared and let, of which two, one for the structural steel frame and the other for the pier doors, were incorporated in the last contract. These contracts are summarized in Table I. It will be noted from this table that all emergency and demolition work, including clearing of the site, was performed between May 12 and June 29 at a cost of $99,768.60, and the substructure reconstruction between July 18 and October 19 at a cost of $255,020.73, or an average of $2.58 per sq ft. The power plant was furnished and installed under a separate contract with the steamship company at a cost of $25,518.

To clear the slip of any old piles, pile butts, or debris occasioned by the driving of the new piles, the engineers conducted very careful and thorough sweeping operations in front of the north and south bulkheads and along both sides of Pier 54 for a width of 80 ft. Three catamaran loads of miscellaneous material were removed and the area cleared to the satisfaction of the underwriters and the steamship company. This work was done between November 18 and December 27 at a cost of $3,656.69. A few minor items not covered in the general contract and subsequently attended to, such as some electric lamps, adjustments in the boiler-room floor, wheel guards, and concrete protection at the base of drain leaders along both sides of the pier shed, amounted to $1,029.64.

To these figures should be added the premium of $6,533.63 on construction bonds, making a grand total of $836,869.27 for all work in connection with the clearing of the site and reconstruction of the pier.

The first ship to make a regular docking at the rebuilt pier was the *Aquitania* on January 10, 1933. The final inspection and acceptance of the work took place on January 18, 1933, two days ahead of the contract time and exactly six months after the reconstruction of the substructure was commenced. During the entire time—a little more than eight months—that the work of demolition and reconstruction was in progress, there was only one accident that could be classed as serious. This, considering the large number of men employed, the nature of the work, and the speed and pressure under which it was performed, seems remarkable and is an indication of the care and skill with which the different contractors planned for, and carried on, the work.

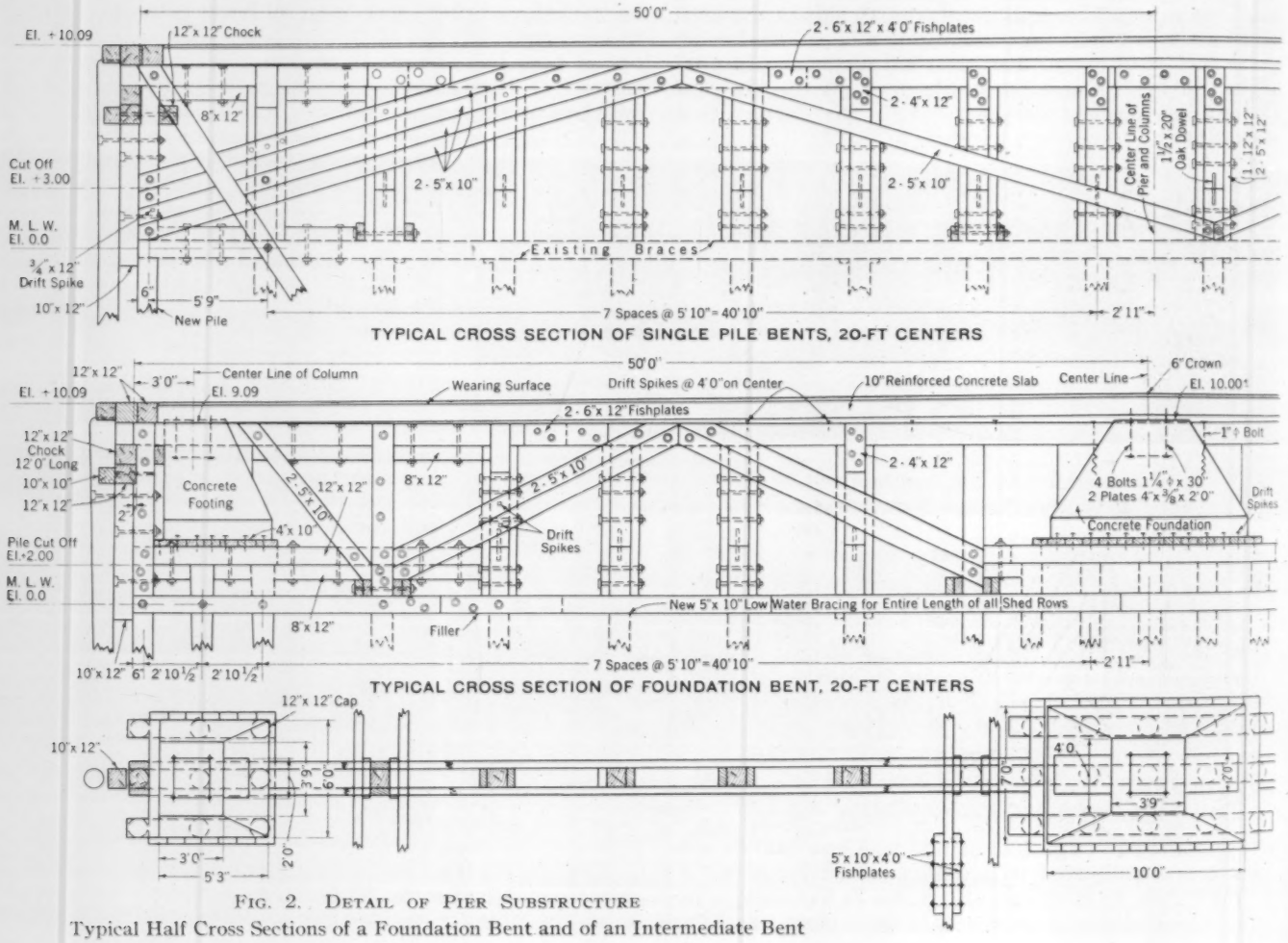


FIG. 2. DETAIL OF PIER SUBSTRUCTURE

Typical Half Cross Sections of a Foundation Bent and of an Intermediate Bent

Value of Water in Southern California

A Historical Résumé of the Cost of Its Development for Irrigation and Domestic Use

By FRANKLIN THOMAS

MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS

PROFESSOR OF CIVIL ENGINEERING, CALIFORNIA INSTITUTE OF TECHNOLOGY, PASADENA, CALIF.

SINCE the founding of Los Angeles in 1781, water in Southern California has become increasingly valuable. Simple diversions from surface streams soon were superseded by wells drawing on ground water in the artesian belts. When these wells ceased to flow, pumping from deep wells was resorted to even for the irrigation of alfalfa and citrus crops. The rapidly growing valley, in which Los Angeles is the largest city, soon found it necessary to seek outside sources to supply the increasing demand for water. First, the melted snow from the eastern slopes of the Sierra Nevadas was conducted to Los Angeles by way of the Owens River Aqueduct, completed in 1913. Now, by the joint efforts of 13 cities in the metropolitan district, construction has been started on the 240-mile aqueduct to convey water from the Colorado River. In this article Professor Thomas shows to what extent the cost of developing water in this area has increased and concludes that water which is needed is worth what it costs.

EXTREME economy in the use of water is necessary in the southern part of California because of the limited quantities available for domestic and irrigation purposes. An equally important reason for such economy is the nature and value of the crops produced. The combination of favorable climate, excellent soil, and good drainage, which is essential to the growth of citrus and other valuable orchard crops, leaves only one need unsupplied, a sufficient amount of water to produce a dependable high return and to maintain exceptionally high values for agricultural land. It is a singular fact that the very valuable orchards and groves not only produce a higher return for the cost of the water supplied, but also require less water ordinarily than field crops of lower value. These factors combine to create a situation where irrigation water is undoubtedly used more efficiently and to produce greater wealth than in any other region of comparable area in this country.

The low comparative availability of water in the southern part of California is strikingly reflected in the following statement by Paul Bailey, M. Am. Soc. C.E., former State Engineer, in Bulletin No. 12 of the California Department of Public Works:

> California southerly from Tehachapi Pass embraces 20 per cent of the area of the state that is favorable for human habitation, while but little over 1 per cent of the state's waters, exclusive of the Colorado River, are tributary thereto.

This insignificant fraction of the waters of the state is at the present time supporting a population of 3,200,000 people (not including Imperial County), representing 57 per cent of the population of California.

ECONOMIC DISASTERS RESULTING FROM DROUGHTS

The early Californians were entirely dependent on surface run-off for their water supply. Whereas much of our modern development has involved the extraction of stored ground waters, the early settlers and their stock were without defense against the hazards of deficient rainfall and run-off. In the two-year drought that began in the fall of 1862 and lasted until the fall of 1864, when it was reported that "not more than a trace of rain" fell during the whole preceding season, the dry feed on the ranges was soon exhausted and the excessively large herds of cattle died by the thousands. One cattle

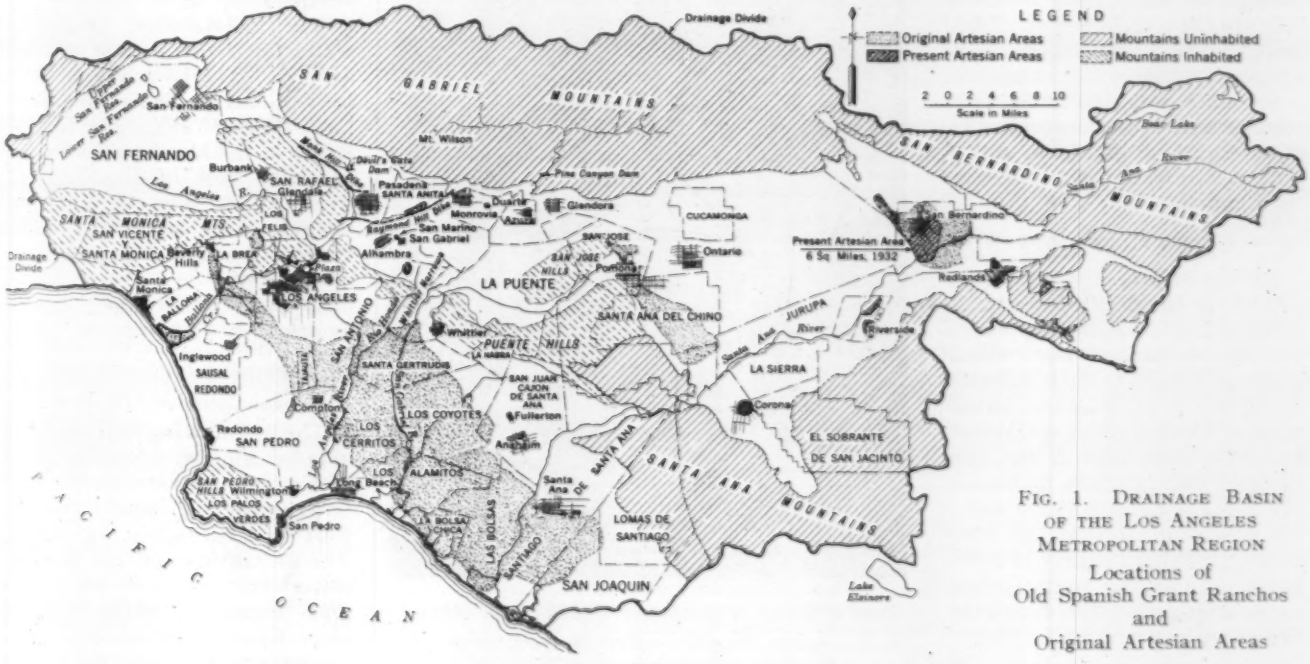


FIG. 1. DRAINAGE BASIN OF THE LOS ANGELES METROPOLITAN REGION
Locations of
Old Spanish Grant Ranchos
and
Original Artesian Areas

baron, Abel Stearns, presiding over a domain of 150,000 acres, including the original Spanish grant ranchos of Los Coyotes, La Habra, San Juan Canjon de Santa Ana, Las Bolsas y Paredes, La Bolsa Chica, and part of the Alamitos, lost 50,000 head of cattle and was reduced to the verge of bankruptcy. His vast landed possessions, shown in Fig. 1, were advertised for sale in 1863 on account of delinquent taxes, which totaled only a little over $2,000.

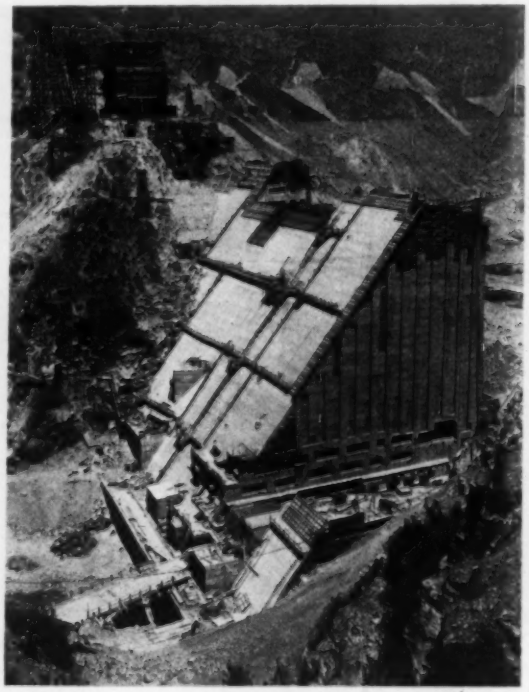
A "CIENEGA" ON THE RIO HONDO AT WHITTIER NARROWS, CALIF. Ground Water Brought to the Surface by a Natural Underground Rock Barrier. Photo by Claude C. Sopp, Assoc. M. Am. Soc. C.E.

This drought during the sixties practically put an end to the cattle industry in the region, and sheep took the place of cattle during the following decade. Then came another long drought, from 1874 to 1877, which resulted in enormous losses to the sheep raisers; 22,000 head died from lack of feed on the Rancho Centinela alone, a part of which is the present site of the town of Inglewood, southwest of Los Angeles.

The next long drought, from 1893 to 1904, placed a severe test on the mechanical means of developing ground water, which had come into general use. A large number of gravity developments of surface water failed entirely in their yields during this trying time. Fortunately, not since the termination of this drought in the winter of 1904 has another such period of deficient rainfall been experienced, although there are ominous precedents recorded in the previous century for the recurrence of extended droughts of much longer duration than this century has seen.

EARLY WATER DEVELOPMENTS

Among the early ambitious efforts for the development and distribution of water supply in the valleys and on the coastal plain adjacent to Los Angeles was a ditch built under the direction of the mission fathers from the west side of the San Gabriel River, where it leaves the mountains, to divert water for irrigation around the Mission San Gabriel nearly 15 miles away. Another outstanding undertaking was the construction, soon after the founding of Los Angeles in 1781, of the *Zanja Madre*, or the "Mother Ditch," extending two miles from the Los Angeles River to a point a short distance northeast of the plaza.



PINE CANYON DAM ON THE SAN GABRIEL RIVER
A Water Supply Project for the City of Pasadena as It Appeared in July 1933

The "Mother Ditch" transported the waters of the Los Angeles River within easy access of the old village, whence water jars could be filled or distribution to the irrigated lands could be effected by means of smaller ditches. Early in the nineteenth century its course was extended through what is now Pershing Square. The "Mother Ditch" of Los Angeles and the laterals which it supplied were in service at the time of the preparation of a report for State Engineer William Ham Hall in 1879, by James D. Schuyler, M. Am. Soc. C.E., then Assistant Engineer. In referring to this distribution system in the town, Mr. Schuyler states:

> They [the ditches] are exceedingly irregular in alignment, passing under the houses, across private property, and only partially following the direction of the streets. They are quite as irregular in grade and in cross-sectional diameter as in alignment. Indeed, the entire distributing system projected and constructed in the fragmentary, careless, and unmethodical manner of the early residents, is of the most primitive character, difficult to maintain in repair, and wasteful of water.

The sources of water supply used by the inhabitants of that day were ditches diverting from streams, *cienegas*, and artesian wells. The *cienegas*—areas where rising ground water caused swamps and marshes to develop—were cited by Mr. Schuyler as representing both the presence of water and the opportunity for easy diversion through drainage ditches. One of the most remarkable features of the country noted at that time was the profusion of springs along the rim of the plateau now defined by the Raymond Dike at the southerly edge of Pasadena, where no less than 18 distinct *cienegas* were counted in a distance of eight miles, each covering from one to 40 acres of ground. Since the springs and even the wells located near them have long since been dry, because of the lowered ground-water level in the basin above, it is of interest to recall the locally significant designations by which these great *cienegas* were known: Bacon's, Stoneman's, Oak Knoll, Wilson's, Shorb's, Winston's, Ford's, Titus's, Rose's, Chapman's, and Baldwin's (three in number, providing water for the irrigation of 1,400 acres). In Mr. Schuyler's report mention is also made of large *cienegas* along the bottom lands of the Los Angeles River; of those west of Los Angeles, from which issued Ballona Creek; of a large one near Pomona; and of two outstanding ones of over 300 acres each, near San Bernardino; as well as others near Riverside.

ARTESIAN WELLS FORMERLY NUMEROUS

The first flowing well obtained in Los Angeles County was bored by Ex-Governor John G. Downey at a point 2½ miles west of Compton, in 1868. The advantages of this form of water supply were so great that numerous other borings were made in the hope of securing similar flows of

artesian water. It was estimated that the average discharge of the wells at that time probably did not exceed 0.1 cu ft per sec, although individual wells discharging from 0.2 to 0.3 cu ft per sec were common, and the one of banner production was the Burlingame Well near Compton, which discharged 1.7 cu ft per sec. The artesian belt on the coastal plain was defined as having a length of 40 miles and a width of from 2 to 12 miles, the total area being about 300 sq miles.

These sources of water supply were utilized by the early settlers almost entirely through gravity flow and involved little expense except for maintenance of ditches and diversion structures. With the discovery of artesian water and the boring of wells, there came the necessity for larger initial costs and the recognition of water as an asset justifying expensive development.

In Mr. Schuyler's report there is a statement of how water values were regarded in 1880:

> If we assume that the average cost of the artesian wells in Los Angeles County has been $400 and that the total number is 550, with an aggregate discharge of 55 cu ft per sec, the total cost foots up to $220,000, or an average of $4,000 per cu ft per sec. When we consider the many failures that have been experienced in seeking for water, it will be seen that the luxury is a somewhat expensive one.

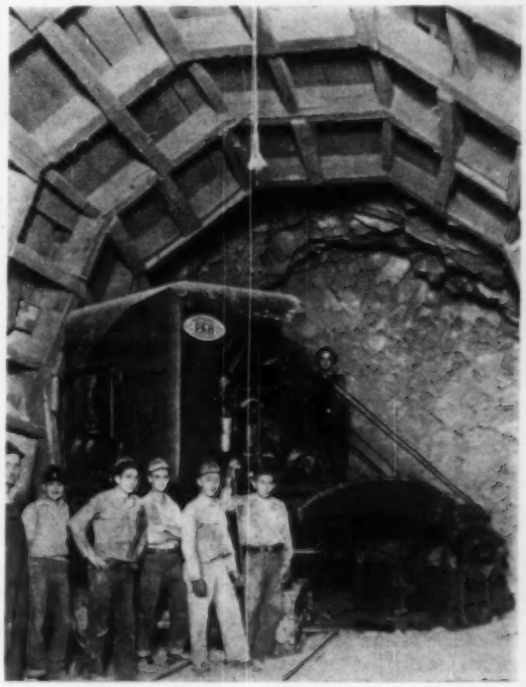
TABLE I. TRANSFER PRICES OF SHARES IN NORTH FORK WATER COMPANY NEAR HIGHLANDS, CALIF.

YEAR	VALUE PER CU FT PER SEC	YEAR	VALUE PER CU FT PER SEC
1865	$ 900	1882 (Apr.)	$ 6,750
1871	1,500	1882 (Nov.)	15,000
1880	1,500	1884	18,000
1882 (Jan.)	4,500	1888	36,000

The same report cites the construction, by the Santa Ana Valley Irrigation Company, of a canal 20.5 miles long, having a theoretical capacity of 114 cu ft per sec. This work was completed in the fall of 1878 at a cost of $49,000, which means a canal cost of $430 per cu ft per sec.

The information presented in Table I relates to the transfers of shares in the North Fork Water Company in the Highlands region east of San Bernardino, as reported on page 151 of Part II of former State Engineer Hall's classic report on "Irrigation in Southern California," dated 1883. In this table the last value of $36,000 for a continuous flow of 1 cu ft per sec in 1888 is very much in excess of the value of $4,000 per cu ft per sec which Mr. Schuyler regarded as high in 1880.

During the decade between 1880 and 1890, notwithstanding several seasons of excessive rainfall, there took place the most pronounced multiplication of values of water that has been experienced in the history of California. Within this period, the possibilities of the locality for producing valuable citrus crops became thoroughly demonstrated. Another factor that greatly increased the demand for water was the land boom of the late eighties based upon the establishment of train service on the Santa Fe Railway.



MUCKING IN COLORADO RIVER AQUEDUCT
East Coachella Tunnel, Horseshoe Section, 16 by 16 Ft, with Capacity of 1,600 Cu Ft per Sec

RISING VALUES OF WATER DURING THE PRESENT CENTURY

In 1903 a proposal was being considered in Pasadena whereby the city would acquire the properties of several of the private companies serving the area within the city. Among the assets of one of the companies was 200

LOS ANGELES AQUEDUCT IN OWENS VALLEY ABOVE THE HAIWEE REGULATING RESERVOIR
Capacity Above Haiwee Reservoir, 900 Cu Ft per Sec; Capacity Below, 440 Cu Ft per Sec

miner's inches (4 cu ft per sec) of water obtained from tunnels at Devil's Gate. It was proposed to allow $1,000 per miner's inch, or $50,000 per cu ft per sec, for this water.

Twenty years later Pasadena acquired the property and water rights of the Precipice Canyon Water Company, including a gravity flow of water from Eaton's Canyon. After deducting the value of the physical property, there remained an allowance of $110,000 per cu ft per sec for the estimated dependable flow of water.

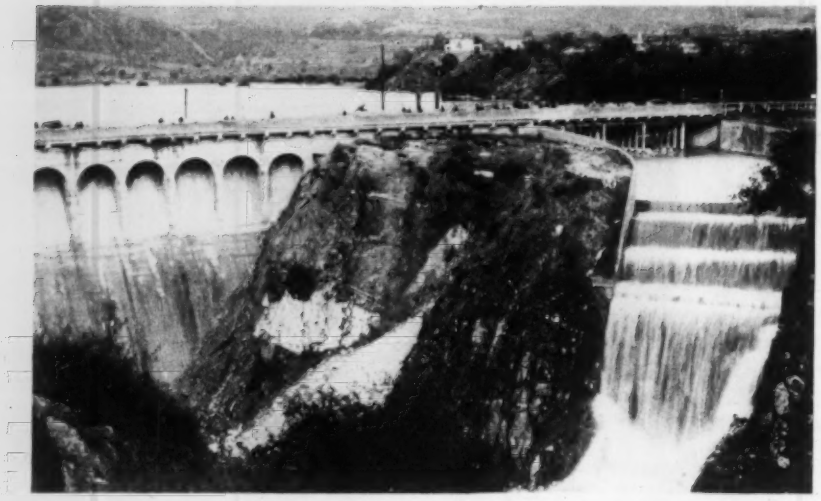
The water produced by the Los Angeles Aqueduct from Owens River Valley, in relation to expenditures made up to 1926, was from an investment of $133,000 per cu ft per sec. This water, allowing for interest and operating charges, without amortization, cost $10,000 per year per cu ft per sec, or $13.80 per acre-ft. Costs as of the present time are practically the same.

The cost of water produced from deep wells of large capacity is illustrated by a typical one put down in the Raymond Basin by the City of Pasadena in 1927. A well drilled to a depth of 1,242 ft, having a 26-in. casing and producing 6 cu ft per sec of water, cost $30,000, or $5,000 per cu ft per sec, including equipment and pump house. The water is lifted 300 ft. The average pumping cost for all the city plants, including power, operation, main-

tenance, complete depreciation in 25 years, and interest at 6 per cent, is 7.6 cents per 1,000 cu ft per 100 ft of lift. The production cost per year, per cubic foot per second for a 300-ft lift, is

$$3 \times \$0.076 \times 31{,}536 = \$7{,}190.21$$

which, capitalized at 5 per cent, equals $143,804—the value of an equivalent continuous flow of water by gravity, less the capitalized value of the annual operation and maintenance charge incidental to producing the gravity water. The production cost per acre-foot per year will equal $\frac{\$7{,}190.21}{724} = \$9.93$.



DEVIL'S GATE DAM, PASADENA, WITH A RESERVOIR CAPACITY OF 4,350 ACRE-FT
A Flood Control and Conservation Project Built in 1919

COST OF PINE CANYON PROJECT

At the present time Pasadena is constructing a dam at the Pine Canyon site on the San Gabriel River to create a reservoir of 40,000 acre-ft. By means of this reservoir, flood waters which heretofore have wasted to the sea will be conserved. The records indicate that the average annual safe yield of the development will be 17,500 acre-ft, or 24.2 cu ft per sec of continuous flow. Estimating that this project will cost $7,500,000, the cost at the service reservoir per cubic foot per second of continuous gravity flow developed reaches the high sum of $310,000.

To calculate the cost of the water from Pasadena's San Gabriel Project on a unit basis the following allowances should be made:

5 per cent interest on $7,500,000	$375,000		
Annual operation and maintenance	50,000		
Total annual charges		$425,000	
Production cost per acre-foot without amortization		$\frac{\$425{,}000}{17{,}500}$	= $24.30
Initial production cost per acre-foot with amortization over 40 years at $187,500 per year		$\frac{\$425{,}000 + \$187{,}500}{17{,}500}$	= $34.70

COLORADO RIVER AQUEDUCT WATER

Although the project of bringing water from the Colorado River is not cheap, it offers the only means of securing a large additional supply. As a matter of fact, the cost will not be out of line with that in a number of large cities and highly developed agricultural districts. Estimating a complete future cost of $250,000,000 for the aqueduct and distribution system, each of the 1,350 cu ft per sec of net delivery would represent an investment of $\frac{\$250{,}000{,}000}{1{,}350} = \$185{,}185$.

The initial unit cost of Colorado River water will be relatively higher as compared to the capital cost than would be the case ordinarily, both because of the pumping charges entailed by a lift of 1,600 ft (although low-priced power generated at Boulder Dam will be used) and because the earlier demands on the aqueduct will be considerably less than its full capacity. Amortization of the 50-year bonds over a 35-year period will commence in 1948, that is, 15 years after date of issue.

The initial cost will be between $40 and $45 per acre-ft on the basis of the following estimated requirements for Colorado River water:

YEAR	CU FT PER SEC
1940	400
1950	725
1960	1,000
1970	1,200
1980	1,350

The initial cost will be reduced as the amount of water served from the aqueduct increases until shortly after 1980, when amortization will be complete. The cost will then be the operating charges only, or about $10 per acre-ft per year.

Thousands of acres of the most valuable citrus lands, especially in foothill areas, have their values of $2,000 to $4,000 per acre supported by some of the most expensive water used for agriculture in this vicinity, the cost of water production being as high as $45 and $50 per acre-ft. Nevertheless the cost of the water is not a controlling factor in the production of a citrus crop and represents only a small proportion of the total cost, which amounts to several hundred dollars per acre.

Itemized costs per acre for orange production in Orange County, in Table II, are taken from Tables 21 and 22 in Bulletin No. 36 of the California Department of Public Works, "Cost of Irrigation Water in California."

TABLE II. AVERAGE COSTS PER ACRE FOR ORANGE PRODUCTION IN ORANGE COUNTY, CALIFORNIA

Including Labor, Materials, and Overhead, 1926–1929

	1926	1927	1928	1929	AVERAGE OF 4 YEARS
Principal labor:					
Irrigation labor	$15.55	$12.20	$17.54	$13.67	$14.74
Cultivation	21.88	16.11	20.71	16.83	18.88
Picking and hauling	32.04	39.87	23.15	45.63	35.17
Principal materials:					
Water	16.68	12.73	11.18	13.70	13.57
Fertilizer	49.14	63.48	62.94	59.95	58.88
Total labor	84.88	98.22	93.40	111.15	96.91
Total materials	73.89	88.04	96.61	92.51	87.76
Cash overhead (general expense, taxes)	31.97	40.28	40.73	41.09	38.52
Total of all cash costs	190.74	226.54	230.74	244.75	223.19
Total computed overhead (interest on property, depreciation on buildings, trees, equipment)	183.80	170.55	177.55	167.67	174.89
Total of all costs	$374.54	$397.09	$408.29	$412.42	$398.08

The average cost of water per acre in the foregoing tabulation is $13.57, which is 3.42 per cent of $398.08, the average total cost per acre for crop production. The amount of water used is between $1^1/_2$ and 2 acre-ft per acre, supplementing an average annual rainfall of from 15 to 20 in. It is pertinent to call attention to the fact that two of the years used in Table II included seasons having considerably more than average rainfall; so that the cost of water in those years is comparatively low

CHARGES FOR DOMESTIC WATER

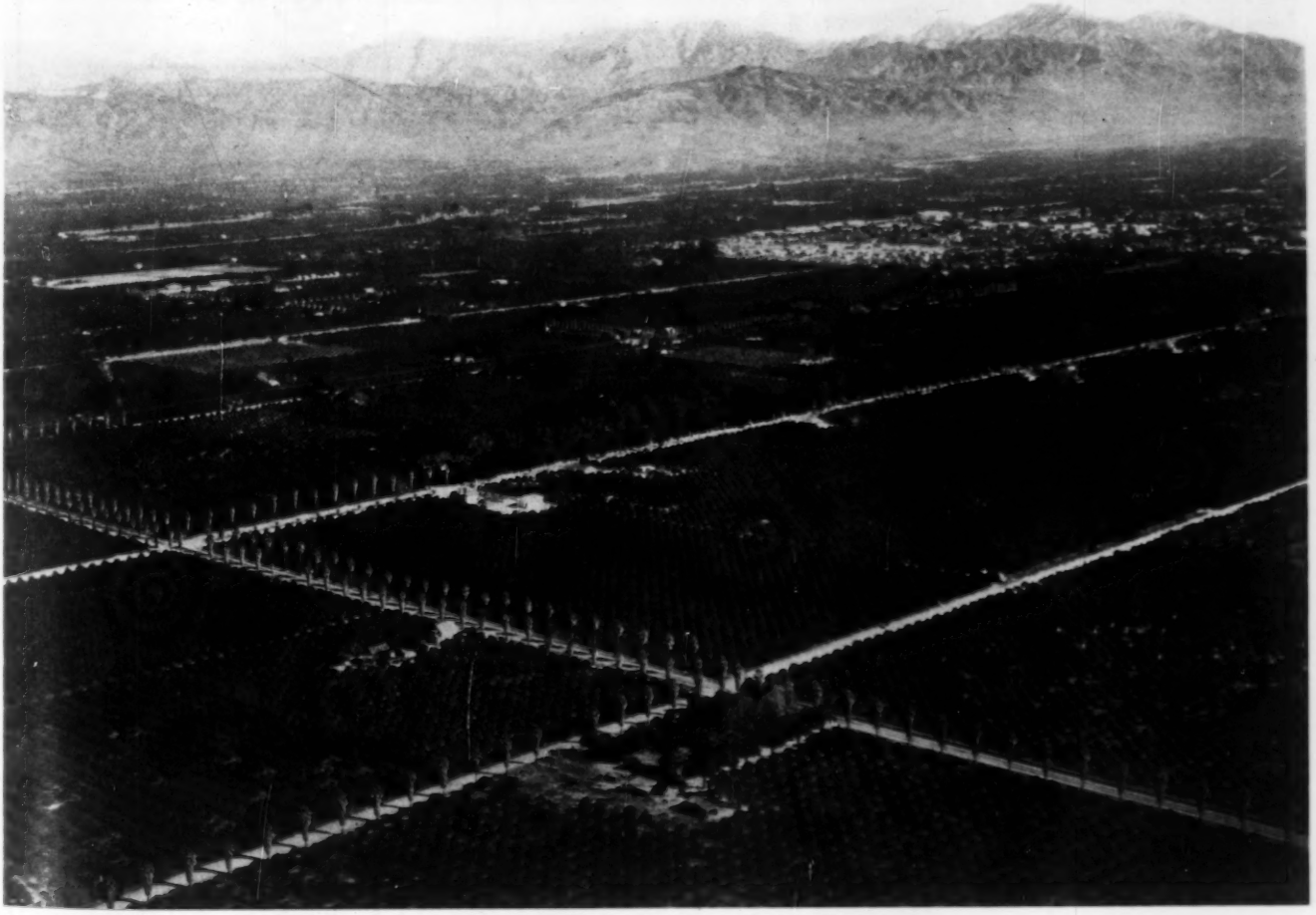
The unit generally employed for the sale of domestic water is that of 100 cu ft per month. Since the acre-foot contains 435.6 such units, at 10 cents per 100 cu ft the retail charge for an acre-foot of water would be $43.56. The large underground water basins of the Los Angeles metropolitan area have provided much water at a relatively low cost, in so far as the increasing demands could be satisfied by this means. Domestic water charges in this vicinity are generally lower than in other populous areas in the state and compare favorably with the charges prevalent in non-arid parts of the country. In Table III are listed the rates charged for domestic water in a number of representative cities.

It is significant that although in the metropolitan area of Los Angeles there are lands for which it has been difficult to obtain the water necessary to continue a high state of development, there has not been, so far as I am aware, any instance in which highly productive land has been abandoned because the cost of water was too high. The evidence points very definitely toward a confirmation of the statement sometimes made that "Water which is necessary is worth what it costs."

TABLE III. RETAIL DOMESTIC WATER RATES IN REPRESENTATIVE AMERICAN CITIES, IN CENTS

Cities with Filtration Plants Indicated by (F); Cities Operating Softening Plants Indicated by (S)

	Rate per 100 cu ft for First 1,000 Cu Ft per Month	Rate on Same Basis for Over 100,000 Cu Ft
California cities:		
Los Angeles	13.0	7.0
San Francisco	36.6	22.0
Oakland	22.0	19.0
Long Beach	15.0	5.0
Pasadena	15.0	13.0
Glendale	13.0	7.0
Beverly Hills (FS)	13.0	13.0
Other cities:		
New York	10.0	10.0
Chicago	6.0	5.1
Philadelphia (F)	10.0	4.0
St. Louis (FS)	14.5	5.0
Baltimore (F)	20.0	7.5
Pittsburgh (FS)	13.5	9.0
Dallas	22.4	15.7
Indianapolis (F)	19.25	6.0
Wilmington (F)	33.5	5.5



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CITRUS GROVES NEAR GLENDORA, CALIF., IRRIGATED BY WATER PUMPED FROM DEEP WELLS

Controlling Floods Along the Mississippi

Recounting Some of the Complex Problems Met with on an Immense Project and Progress to Date

By T. B. LARKIN

MEMBER AMERICAN SOCIETY OF CIVIL ENGINEERS

MAJOR, CORPS OF ENGINEERS, U.S. ARMY, VICKSBURG, MISS.

TODAY flood-control work on the Mississippi constitutes one of the major construction activities of the country. Because it is being prosecuted efficiently and quietly, engineers are apt to overlook its importance. Actually, levee construction is advancing at the rate of 100,000,000 cu yd per year and the total expenditure over a 10-year period will amount to $325,000,000. As Major Larkin here explains, much has been and is being learned about the causes and cures of subsidence, leakage, scour, and caving. This article, giving an intimate picture of the many problems engaging the attention of the engineers on this huge project, has been prepared from an address delivered by Major Larkin before the Mid-South Section of the Society at Vicksburg, Miss., on May 5, 1933.

TWENTY million acres comprise the alluvial valley (Fig. 1) of the Mississippi, the world's largest river. It drains 31 states and varies in flow between approximately 71,000 cu ft per sec just below the Ohio River at extreme low water, to a maximum discharge of approximately 2,000,000 cu ft per sec between the levees of the main river. Prior to the great deluge of 1927, there was a widely accepted theory that the floods could be confined between riverside levees. After 1927, however, it was recognized that in times of extraordinary floods the excess waters must be permitted to escape at certain localities more or less definitely predetermined.

The solution as recommended by the Chief of Engineers and adopted by the Act of Congress of May 15, 1928, is a comprehensive plan for the protection of the alluvial valley. Its main features include the following:

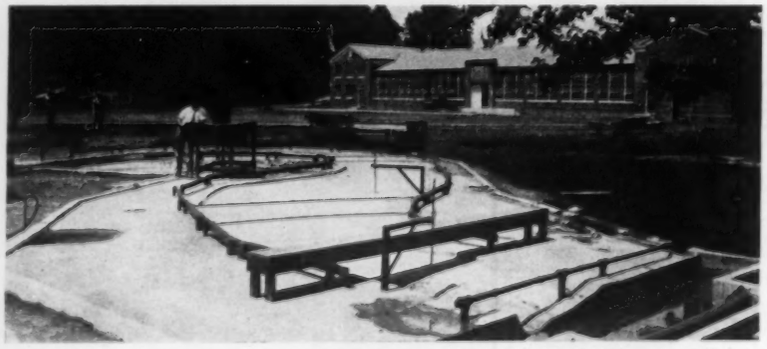
1. Existing levees, with certain exceptions, are to be raised about 3 ft and strengthened greatly.

2. Because of the bottle neck at Cairo, where the Ohio empties into the system, additional local safety is to be provided by a floodway between Birds Point and New Madrid, Mo. During extraordinary floods only, this will become an adjunct of the main channel and will thereby widen the river and hold down high stages. The existing levees on the west bank between Birds Point and New Madrid are to be retained but lowered about 3 ft.

3. South of the Arkansas River, flood waters in excess of those which the levees of the main river will safely carry will find their way to the Gulf through the lowlands of the Boeuf and Atchafalaya basins west of the river, where the excess waters of extraordinary floods have usually escaped during the past. The diversion is to be accomplished by leaving the existing levees above and below Arkansas City and at the head of the Atchafalaya Basin at their present grade and section.

Protecting levees are to be constructed on the edges of the lowlands in the Boeuf and Atchafalaya basins so as to restrain the waters within them and protect the good lands outside. The protecting levees are located so as to reclaim as much land as practicable. The flood waters will not spill into the side lowlands until they have reached a dangerous stage in the main river channel, so that the lands between the protecting levees in the side basins will be subject to overflow only during extraordinary floods. In general, they will retain levee protection equal to or greater than what they have had in the past. The use of these lands can be continued, but they will be subject to overflow during extraordinary floods, which occur on the average not more often than once in 10 or 15 years.

4. The City of New Orleans is to be given additional protection by a controlled spillway at Bonnet Carré emptying through a floodway into Lake Pontchartrain.



MODEL OF HOTCHKISS BEND (MILE 59 TO 67)
Used at the U.S. Waterways Experiment Station to Study Effects of a Dike

HOW FLOODS ARE DISTRIBUTED

This plan may be presented in another way, by showing the quantitative disposition of the waters, as follows:

1. Between Cairo and the Arkansas River at high stages there will be from 2,250,000 to 2,400,000 cu ft per sec, of which about 450,000 cu ft per sec will be carried by the New Madrid Floodway between Birds Point and New Madrid, where it will be returned to the main channel.

2. Between the Arkansas and Red rivers, 1,950,000 cu ft per sec will be carried by the main river and about 1,000,000 cu ft per sec will escape through the Boeuf Floodway.

3. Below the Red River, 1,500,000 cu ft per sec will be carried by the main river and an equal amount will escape down the Atchafalaya River and floodways.

4. At New Orleans, the main river will carry 1,250,000 cu ft per sec, and 250,000 cu ft per sec will pass through the Bonnet Carré Spillway 30 miles above New Orleans, thus holding the stage at Carrollton at or below 20 ft.

This plan gives complete protection—barring unexpected accidents—to about 13,000,000 of the 20,000,000 acres in the alluvial valley. The remaining 7,000,000 acres, which will remain subject to overflow in varying degree, include backwater areas and sec-

tions into which the waters will escape at extremely high stages.

The Flood Control Act authorized $325,000,000 for flood protection in the Mississippi Valley. This includes the cost of the levees, revetment work, and contraction and dredging for river stabilization so as to provide a navigable channel 9 ft deep and at least 300 ft wide, together with certain minor auxiliary expenses. The work, to be completed in 10 years after the passage of the act, or by May 1938, is being carried on under Lytle Brown, M. Am. Soc. C.E., Chief of Engineers, U.S. Army; Gen. Harley B. Ferguson, president of the Mississippi River Commission, whose headquarters are at Vicksburg, Miss.; and the three Engineer Districts, Memphis, Vicksburg, and New Orleans.

To accomplish this immense task it will be necessary to build about 1,900 miles of levees (Fig. 1), of which 1,100 miles have already been placed or are under contract. The aggregate volume of earth involved reaches the stupendous total of almost 650,000,000 cu yd. But there is much more to levee building than placing a pile of dirt. In the first place, a most careful study must be made to determine whether the proposed foundation will stand up under the load.

Preliminary foundation borings are made along the center line of a proposed new levee and along the riverside toe of an old levee whose enlargement is contemplated. These borings are made at 500-ft intervals to a depth of 25 ft, with a 4 or 6-in. post-hole digger. Where caving strata are encountered, a 4-in. casing and a 3-in. post-hole digger and sand bailer are used. In all sloughs and in places where the preliminary borings show blue or lake mud, borings are made at 100-ft intervals to depths well into the blue mud, and several are driven through the blue-mud stratum. The top strata are classified according to the standard method of the U.S. Bureau of Soils, and two samples, taken well into the blue mud, are carefully tested to determine their resistance to lateral flow and vertical and horizontal compression. These results are used in designing the section so as to ensure the construction of a stable levee over a given foundation. Borings are made every 1,000 ft in the proposed borrow pits to determine the type of section necessary to prevent sloughing or sliding and excess seepage.

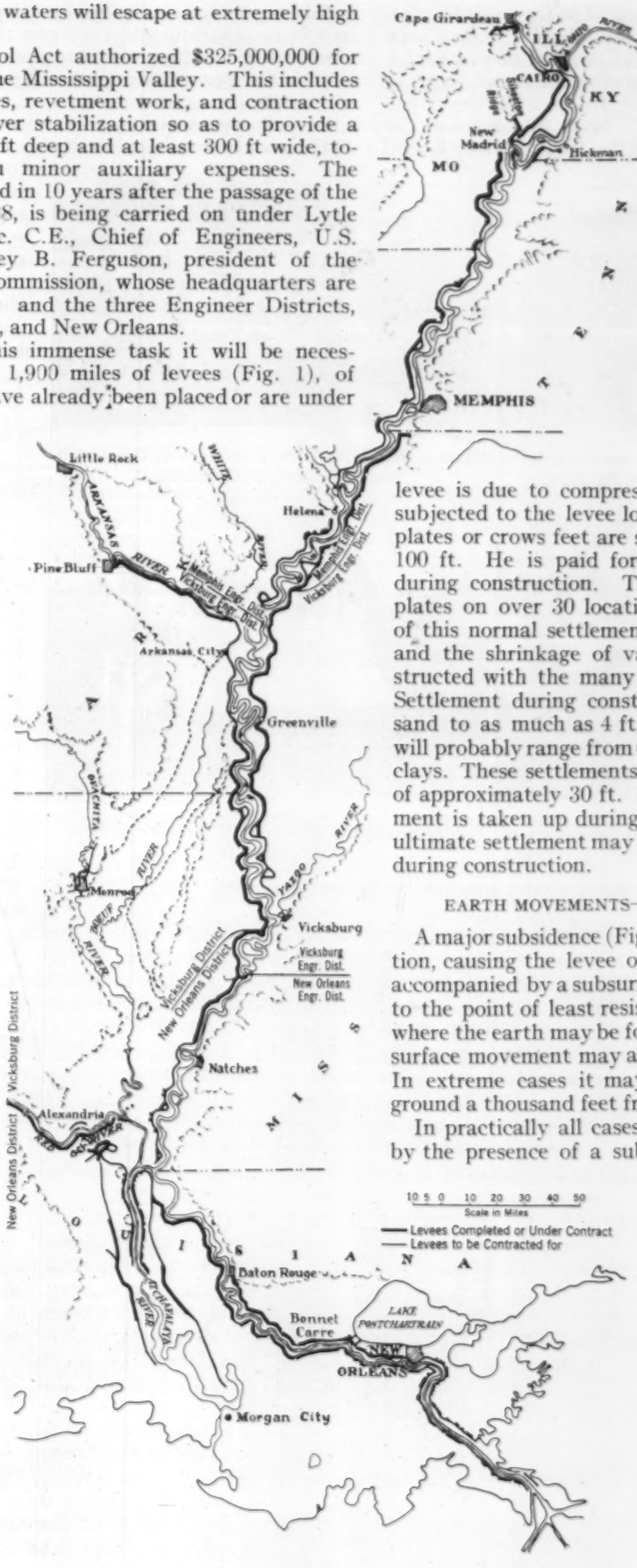


FIG. 1. THE ALLUVIAL VALLEY OF THE MISSISSIPPI RIVER
Progress of the Flood Protection Project as of April 1933

Normal settlement of a levee is due to compression of the foundation when subjected to the levee load. To determine its amount, plates or crows feet are set up by the contractor every 100 ft. He is paid for the settlement which occurs during construction. The Vicksburg District has set plates on over 30 locations to determine the amount of this normal settlement for various types of material, and the shrinkage of various types of material constructed with the many different types of equipment. Settlement during construction ranges from 0.25 ft in sand to as much as 4 ft in poor clays. The ultimate will probably range from 0.5 ft in sand to 6 or 8 ft in poor clays. These settlements are for levees with a gross fill of approximately 30 ft. In sands, most of the settlement is taken up during construction, but in clays the ultimate settlement may be several times the settlement during construction.

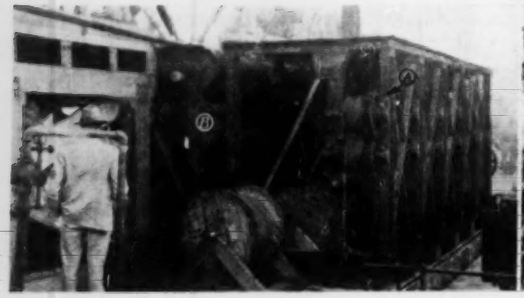
EARTH MOVEMENTS—SUBSIDENCE OR SLIDING

A major subsidence (Fig. 2) is a rupture of the foundation, causing the levee or a part of it to drop, and is accompanied by a subsurface movement of the material to the point of least resistance, generally a borrow pit, where the earth may be forced up several feet. The subsurface movement may also cause a raising of the berm. In extreme cases it may move trees and disturb the ground a thousand feet from the levee.

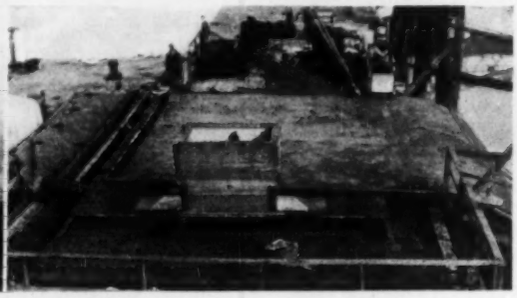
In practically all cases, major subsidences are caused by the presence of a substratum of soft blue mud or soft clay, deposited in old lakes formed by cut-offs of the Mississippi River and now overlain with a top stratum of clay or loam. The depth of this overlying stratum is governed by the age of the lake and its location with reference to the river. In some cases, these lake beds are so old that there are no surface signs of them, and they are found only by careful boring.

A general vertical settlement of the levee due to the compression of this soft mud cannot be economically prevented, but the mud can be so confined that lateral

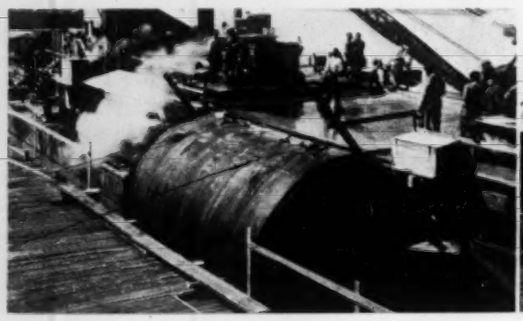
flow will not occur and the blue mud in time will be compressed to its limit by the applied load. It has been found that the most economical method of preventing or correcting a major subsidence is by the use of berms as counterweights. The Vicksburg District has developed a method of berm design which takes into account the applied load, the resistance of the blue mud to lateral flow, and the resistance of the top stratum or berm against upheaval.



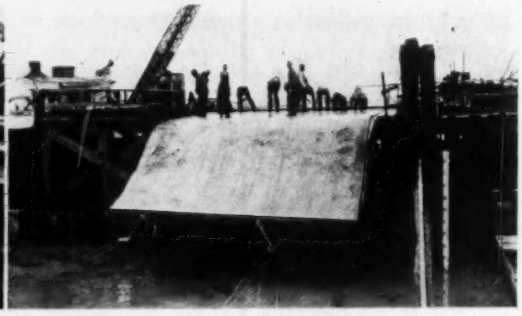
(*A*) Figure-Eight System of Launching Cables; (*B*) Brakes to Permit Tightening of Reinforcing



Casting or Pouring Deck Covered with Completed Mattress Ready for Launching



Experiment to Measure Frictional Resistance of Mattress to Starting and Subsequent Movement



Mattress Being Hauled to Water's Edge From Pouring Deck

STEPS IN PREPARATION AND LAUNCHING OF ASPHALT MATTRESS

It has been found by experience that a false berm constructed beforehand to prevent subsidence (Fig. 2) need be only about half the size required after the subsidence occurs. When the blue mud is confined by a berm, the subsidence is proportional to the height of the levee (that is, to the applied load), the type of the blue mud, and its depth. The maximum subsidence that has occurred when the blue mud was confined was about 40 ft. In this case the blue mud was at a depth of 80 ft and the gross fill was 40 ft.

A slide (Fig. 3) is a downward and lateral movement of a part of the levee, resulting in the formation of a smooth plane of cleavage between the moving and fixed masses of earth. It may vary in length along the levee from 30 to over 1,000 ft. The principal causes are steep construction slopes of partial buckshot or clay fills, the placing of wet buckshot or clay in the section, or the placing of dry material on buckshot fills that have become wet. A slide may be either landward or riverward, but is generally toward the river, since the levee slope on that side is steeper.

In all cases of sliding the only correct and satisfactory cure is to remove the planes of cleavage. Slides not properly corrected will be a continual source of weakness and danger in a levee system. High water lubricates cleavage planes and increases the load above them, causing the material to slide out when the river falls. Since at such times it is almost impossible to procure material for protecting the weakened section, the situation may be most serious and a crevasse may result. Furthermore, the contractor who is responsible for the slide will have been released and thus will escape the penalty for such improper construction.

On first consideration it sometimes appears difficult to distinguish between a slide and a major subsidence, but to the experienced levee engineer the differences are quite apparent. A slide shows a smooth plane of cleavage, but in a subsidence the shearing surfaces are fractured like a broken brick. The location and shape of the surface cracks in the levee are also different. Cross sections taken before and after a slide show no change in area, but do show a reduction in the case of a subsidence.

DRAINAGE PROVIDED

In the course of levee construction, local drainage is sometimes interrupted. Under the Flood Control Act of 1928, local interests are required to provide for drainage. In this connection, the works recently installed in Black Bayou at West Monroe, La., by that city, may be of interest. After some study and research, two corrugated galvanized iron pipes, No. 8 gage, 5 ft in diameter, with flood gates, were recommended and constructed. Corrugated diaphragms, or cut-off collars, 8 ft square were provided at 20-ft intervals to prevent seepage along the pipe. The pipe and diaphragms were fully coated with a special asphalt $^1/_{16}$ in. thick. In addition, the invert was paved with this asphalt along about one-sixth of the circumference to a depth sufficient to fill the corrugations. This coating provided a considerable thickness of tough, non-erodible medium to protect the corrugations against scour.

Corrugated pipe was selected in preference to a rigid drain for the following reasons:

1. All levees are subject to more or less settlement of the foundation. Corrugated pipe, being flexible, is designed to absorb this settlement without injury.
2. Rigid structures, such as concrete pipe, require expensive foundations to prevent settlement. If settlement occurs, they crack, and the levee is endangered from seepage.
3. The average loads on flexible structures are much less than on rigid ones since any settlement of the structure transfers the load laterally through the structure to the soil beneath, and thus permits arch action in the soil to relieve direct loading on the structure.
4. The corrugated pipe design is safer, more economical, and more easily repaired.

On the diaphragms the corrugations are vertical and take up any expansion or contraction. For instance, the

compression of the pipe along its vertical axis causes a corresponding elongation along the horizontal axis. The pipe forces a point on the diaphragm, halfway between the top and bottom of the band, to move outward, and a point at the top of the band to move downward. When installed, the band is drawn tightly around the pipe and takes the same deflection as the pipe. The diaphragm is electrically welded to the band and thus cannot have any motion along the band. The circumference of the band and pipe does not change even though the pipe deflects; consequently the welded joint will not rupture. However, due to the flexibility of the corrugated sheets, all other parts of the diaphragm can be compressed or elongated wherever necessary.

At West Monroe the levee was constructed with tractors. The maximum increase in the 5-ft horizontal diameter was $2^1/_4$ in. at the diaphragm under the tractor run, while the maximum increase in diameter at the center of sections of pipe under the tractor run was $4^5/_8$ in. The maximum deflection of the pipe under the center of

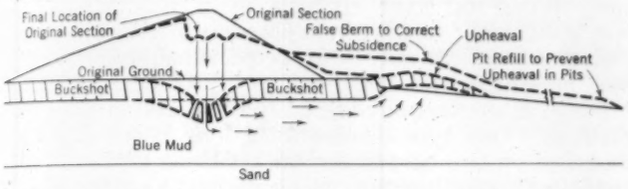


FIG. 2. SECTION OF A MISSISSIPPI LEVEE ILLUSTRATING A MAJOR SUBSIDENCE

the levee, where the fill was 30 ft, amounted to only $1^3/_4$ in. The asphaltic coating is elastic enough to bend with the metal and can be easily repaired if injured during assembly of the pipe in the field.

Another major item provided for by the adopted project is the protection of caving river banks by revetment. Revetment consists of a flexible, continuous mat laid along the river bank and bottom, together with paving on the upper bank to protect it from attack by the current.

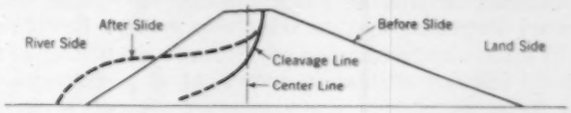


FIG. 3. FEATURES OF A TYPICAL SLIDE IN A MISSISSIPPI LEVEE

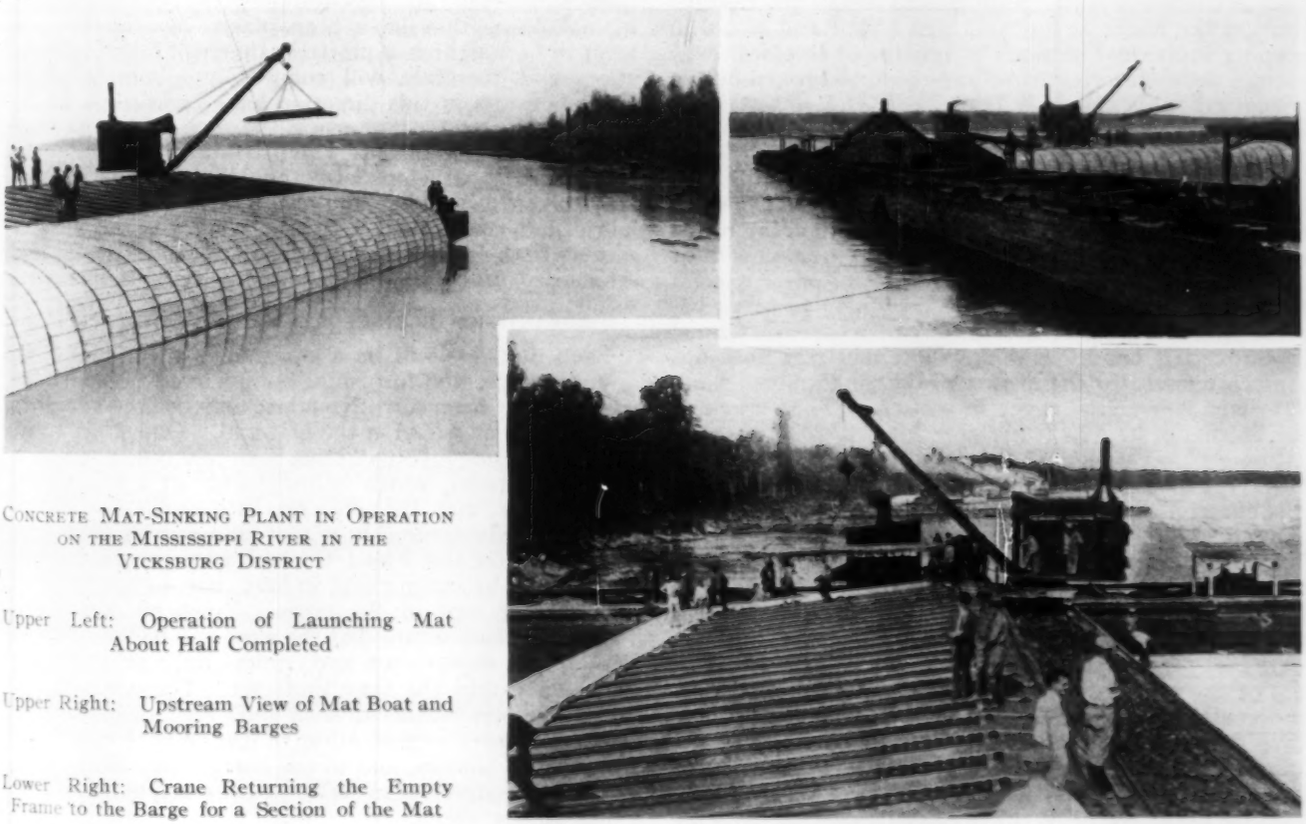
However, the immediate and direct purposes of revetment may differ widely. Some of its more important uses are as follows:

1. To prevent a cut-off at the neck of a deep bend.
2. To prevent undue widening by erosion on the bank opposite contraction works.
3. To protect towns.
4. To protect levee banks from caving, if economically justified.

Revetment costs about $350,000 per mile while levees cost on an average about $110,000 per mile, so that the use of revetment is restricted to special cases, such as safeguarding of levees which protect wealthy and populous sections, or those in close proximity to a lake, where the failure of the levee would inundate a large area.

Revetment serves another useful purpose. Since it prevents caving, it reduces the volume of silt that the river must carry. In the Memphis District the banks are more friable and the river slopes steeper than in the Vicksburg or New Orleans districts. Consequently, the Memphis District must place more revetment than either of the other two. The total constructed during the last five years is about 141 miles.

Occasionally caving banks may occur on straight



CONCRETE MAT-SINKING PLANT IN OPERATION ON THE MISSISSIPPI RIVER IN THE VICKSBURG DISTRICT

Upper Left: Operation of Launching Mat About Half Completed

Upper Right: Upstream View of Mat Boat and Mooring Barges

Lower Right: Crane Returning the Empty Frame to the Barge for a Section of the Mat

reaches but they are found principally at bends. The rate normally varies with the degree of curvature of the bend and the friability of the bank. When the concave bank is held by protection works the river tends to deepen its channel in lieu of cutting away the bank. This channel deepening may be relatively great. Revetment works must also be planned so that such deepening of the channel will not ultimately destroy their usefulness.

TWO STANDARD TYPES OF MATTRESSES NOW IN USE ON THE MISSISSIPPI

At present, subaqueous mattresses on the Mississippi River are generally of two standard types, willow and articulated mat. Paving may be of riprap, stone, or concrete.

The construction of revetment involves four distinct operations: bank clearing, bank grading, placement of the subaqueous mattress, and paving of the upper bank. Clearing must be done not only on the bank itself but also on a strip behind the bank and beyond the top of the bank paving. In this operation all trees and stumps likely to impede or reduce the efficiency of the work, or to cause snags in the river, are removed.

Bank grading is an important feature for several reasons. First, its primary purpose is obviously to prepare the upper bank for paving. Second, it removes unnecessary weight from the upper bank to an amount determined largely by the stability of the material comprising the bank. However, no banks are graded to slopes steeper than 1 on 3. Third, it removes all projecting points and irregularities in the bank line, thus decreasing turbulent flow and eddy action and making possible the placing of successive parallel mattresses with the minimum waste due to overlapping. Fourth, the most important and most difficult to accomplish, it grades the bank to a suitable slope from the water surface to the bottom of the bank in the river. The purpose of the under-water grading is to reduce the tractive force of the water at low and high stages and to aid in keeping individual sections of revetment in place even though the connecting wires have become broken or disconnected.

PROBLEMS NUMEROUS IN REVETMENT WORK

All willow mattresses are particularly vulnerable to attack at two points, the water surface and the toe of the revetment. At the water surface, the rapid decay of the willows due to alternate wetting and drying necessitates expensive maintenance, and at the toe, or outer edge of the revetment, undermining is very likely to occur because of the river's normal tendency to deepen its channel in a revetted bend. A stiff willow mattress does not conform closely to the shape of the undermined bank but extends over the caved-in section without touching the bottom of it.

Concrete revetment has certain advantages over willow. It is cheaper both in first cost and in maintenance. The effective under-water weight of the concrete mat in place is about 18 lb per sq ft as compared to about 5 lb for a willow mattress covered with 2.5 tons of riprap per 100 sq ft. This makes the concrete mat the more suitable of the two. Flexibility is the second great advantage, since the articulated formation of the concrete mat permits it to conform closely to the irregularities of the river bank and bottom and ensures against undercutting at the toe. Properly designed, it should have a longer life than a willow mattress because the concrete does not decay and the metal ties between the units are comparatively well protected.

In the under-water survey of 1931, divers examined the revetment all the way out to the toe. This survey showed that the attack of the current was more severe than had been assumed and that galvanic action had occurred and quickly destroyed common steel and galvanized fabric and connections. Their failure naturally hastened the failure of the revetment itself.

The problems in revetment may be summed up by the simple statement that initial cost and maintenance are too high. With a view to decreasing the initial cost and especially to increasing the life and effectiveness so as to reduce the maintenance cost, intensive studies have been made of revetment in the three districts during the past two years. Present specifications require for fabric and connections the use of metals having the same resistance to corrosion as pure copper. There are a number of homogeneous bronze and bimetallic wires which meet these specifications.

More care and attention have been given to grading both upper and subaqueous banks. Formerly bank grading was done by hydraulic monitors. But by this method the unit cost was relatively high; deep gullies were often left in the bank; and it was impossible to grade below the water surface. As may be expected, subaqueous grading has presented the most difficult problems. Various types of equipment have recently been used in the three districts. One type consists of a long boom drag-line mounted on two barges, the bucket being replaced by a spade with long handles, which moves along the boom. Another type consists of a tower levee machine with the head tower on a barge and the tail tower on the bank. This permits of a long span and more efficient subaqueous grading. In order to prevent undermining at the toe due to deepening of the channel, revetment is now carried out beyond the thalweg of the stream, that is, from 300 to 400 ft instead of 200 or 250 ft as previously.

Annual surveys and studies, together with the results of the under-water survey of 1931, show that scour proceeds at varying rates through the interstices of all types of mattresses that have been placed heretofore. The ideal to be sought is a mattress that will have no interstices and therefore will completely eliminate scour. Improvements in this direction have been made in the articulated mat by decreasing the size of the openings between the unit blocks. The reinforced asphalt mattress recently developed by the New Orleans District may solve this problem and also that of the corrosion of fabric and connections. Recent tests give promise of success both from the point of view of effectiveness and economy.

ADVANTAGES OF REINFORCED ASPHALT MATTRESS

Each mattress will be a continuous sheet, free from any interstices and furnishing complete coverage for the river bed. Cheap corrosive fabric may be used since it is completely embedded in the material. One of the problems of concrete revetment has been to secure a sufficient supply of gravel, which is found only at a few places. In the asphalt mat, the only aggregate used is sand, which can be readily secured at any of innumerable sand bars.

Progress on the Flood Control Project can be best summed up by stating that in four years of construction over 60 per cent of the project has been completed. Present indications are that the work will be finished well within the project cost and, unless appropriations are cut, well within the time limit set. These results are due to the performance of each and every member of the Army Engineer organization of the War Department from top to bottom, and to the energy and efficiency of American contractors as well as to the competition that exists among them.

The Hydraulic Jump in Standard Conduits

General Formula for Rectangular Channels Extended to Trapezoidal and Curvilinear Sections

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While engineering literature contains considerable information on the characteristics of the hydraulic jump in ordinary rectangular channels, considerably less is to be found concerning the calculations required when the jump forms in conduits such as circular flumes, trapezoidal canals, or triangular channels. In order to calculate the jump in such sections, Mr. Stevens has simplified the derivation of complicated formulas to a marked extent. His formulas for height and energy loss are supported by sample calculations and tables which should prove of value to designers.

MOST writers on the hydraulic jump have confined their treatment of it to a unit width of a wide channel. The characteristics of the jump in standard types of conduits follow logically from a generalized treatment of the simple form of the usual formula. The characteristics of the jump are obtained by use of the momentum and energy equations which have been quite exhaustively presented.

In the short length in which the jump occurs in a uniform conduit, all forces except the pressure in the cross sections of the water prism may be safely ignored. The general equation for momentum then becomes:

$$P_1 - P_2 = \frac{Q}{g}(V_2 - V_1) \ldots\ldots [1]$$

where P is the pressure in cubic units of the liquid, and V is the velocity. The left-hand term represents the change of pressure on a mass of water bounded by the cross sections numbered 1 and 2, corresponding to the subscripts of P and V. The right-hand member expresses the change of momentum of that mass as it passes those sections. Section 1 is taken just above the jump and Section 2 just below it. In any right-angle section of a regular prism the cross-sectional area, a, may be expressed as a function of the depth, y; that is, $a = f(y)$. It may be shown that the hydrostatic pressure in the same cross section is given by the equation,

$$P = \int a\, dy \ldots\ldots [2]$$

That is, the hydrostatic pressure in terms of a cubic unit of the liquid is the integral of the area with respect to the depth. For constant flow, Q equals av, from which it follows that

$$V_2 = \frac{a_1}{a_2} V_1 \ldots\ldots [3]$$

Substituting Equations 2 and 3 in Equation 1 and using h to designate the velocity head, Equation 1 becomes

$$2a_1 h_1 (a_1 - a_2) = a_2 \int (a_1 - a_2)\, dy \ldots [4]$$

which is a general expression of the momentum relation in any type of conduit in which cross-sectional areas may be expressed as a function of depth.

Let $r = \frac{h_1}{y_1}$, the ratio of velocity head to depth before the jump.

$J = \frac{y_2}{y_1}$, the ratio of depths above and below the jump, or the height of the jump in terms of the depth above it.

Expressions for the height of the jump, in terms of the depth and velocity head upstream from the jump, may then be found for many standard conduits.

HEIGHT OF JUMP

For a rectangular conduit, the area, a, equals $b\,y$, where b is the bed width. The pressure in a cross section of the water prism is

$$P = \int b\, y\, dy = \tfrac{1}{2} b\, y^2 \ldots [5]$$

Substituting in Equation 4 it is found that

$$J^2 + J - 4r = 0 \ldots\ldots [6]$$

which is the same as for a unit width of a wide channel.

By a similar process, the following formulas may be derived. For a triangular conduit, as shown in Fig. 1 (*a*)

$$J^4 + J^3 + J^2 - 6r(J + 1) = 0 \ldots [7]$$

For an exponential conduit, that is, any conduit in which the area a may be expressed as equal to $K y^n$, where K is a determinable constant.

$$\frac{J^n (J^{n+1} - 1)}{J^n - 1} = 2r(n + 1) \ldots\ldots [8]$$

For $n = 1$ this equation reduces to Equation 6, and for $n = 2$ it reduces to Equation 7.

In a parabolic conduit, as shown in Fig. 1 (*b*), the general expression for the area, a, of the section in terms of depth is

$$a = K y^{3/2} \ldots\ldots [9]$$

from which K may be determined from any known depth and area. The height of the jump in such a conduit is therefore given by Equation 8 for $n = \frac{3}{2}$, which becomes

$$J^4 - (5r + 1) J^{3/2} + 5r = 0 \ldots [10]$$

The general case of a trapezoidal section in which the side slopes have different values is shown in Fig. 1 (*c*). Using s for the average of the side slopes, the general expression for the area is

$$a = b\, y + s\, y^2 \ldots\ldots [11]$$

from which the height of the jump is found by the equation

$$12r(b + s\, y_1)[b + s\, y_1(1 + J)] - J(b + s\, y_1 J)[3b(1 + J) + 2 s\, y_1 (1 + J + J^2)] = 0 \ldots [12]$$

If s equals zero this reduces to Equation 6 for a rectangular section, and if b equals zero it reduces to Equation 7 for a triangular section. For any particular case in which the initial values of r_1, b_1, s_1, and y_1 are known,

Equation 12 readily reduces to a fourth-degree equation in the form,

$$J^4 + AJ^3 + BJ^2 - CJ - D = 0 \ldots\ldots [13]$$

The substitution of t for $\frac{b}{s\,y_1}$ in Equation 12 will permit its reduction to the dimensionless form, in terms of J, r, and t.

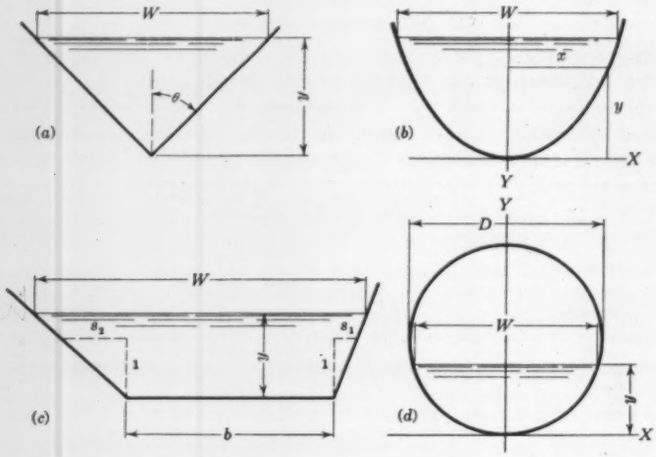


FIG. 1. CROSS SECTIONS OF STANDARD CONDUITS

The mathematical derivation of the formula for a circular section is more involved than that for any of the other sections.

The equation of the circle, referred to the axes shown in Fig. 1 (*d*), is

$$x = \sqrt{2\,R\,y - y^2}$$

The top width, w, of any segment equals $2\,x$, and the area, a, of any segment is

$$a = 2\int \sqrt{2\,R\,y - y^2}\,dy$$

For convenience in integrating, the origin is moved to the center of the circle by writing $y = z + R$. Then this integral is substituted in Equation 4. By integrating each factor and combining, the result obtained is

$$P = (z^2 + 2\,R^2)\,\frac{1}{3}\,\sqrt{R^2 - z^2} + R^2 z \sin^{-1}\frac{z}{R} + \frac{\pi}{2}\,R^2 z \ldots\ldots [14]$$

In this case the constant of integration is zero.

The origin can now be changed back to the base by writing $z = y - R$. It will also be convenient to express depths as a percentage of the diameter by writing $y = Z\,D$; whence $z = \frac{D}{2}\,(2\,Z - 1)$. Also, $\frac{D}{2}$ is substituted for R, and the angle function is expressed in degrees rather than radians, with the result that from Equation 14 the expression for the pressure in a segment of a circle is finally obtained, as follows:

$$P = D^3 \left\{ \frac{(2\,Z - 1)^2 + 2}{12}\,\sqrt{Z(1 - Z)} + 0.00218\,(2\,Z - 1)\,[\sin^{-1}(2\,Z - 1) + 90^\circ] \right\} \ldots [15]$$

The pressures in cubic units of the liquid, in segments of circles whose center depths are given in terms of the diameters of the bounding circles, are listed in Table I.

TABLE I. HYDROSTATIC PRESSURE IN SEGMENTS OF CIRCLES IN CUBIC UNITS OF THE LIQUID

Given $y = ZD$; to Find Pressure Multiply the Tabular Values by D^3

Z	$\frac{P}{D^3}$	Diff.	Z	$\frac{P}{D^3}$	Diff.	Z	$\frac{P}{D^3}$	Diff.
0.00	0.00000	..	0.35	0.0356	23	0.70	0.1815	59
0.01	0.00002	2	0.36	0.0381	25	0.71	0.1875	60
0.02	0.00005	3	0.37	0.0407	26	0.72	0.1935	60
0.03	0.00011	6	0.38	0.0434	27	0.73	0.1996	61
0.04	0.00020	9	0.39	0.0462	28	0.74	0.2058	62
0.05	0.00033	13	0.40	0.0491	29	0.75	0.2120	62
0.06	0.00051	18	0.41	0.0521	30	0.76	0.2183	63
0.07	0.00073	22	0.42	0.0552	31	0.77	0.2247	64
0.08	0.00101	28	0.43	0.0584	32	0.78	0.2312	65
0.09	0.00133	32	0.44	0.0617	33	0.79	0.2378	66
0.10	0.00170	37	0.45	0.0650	33	0.80	0.2446	68
0.11	0.00216	46	0.46	0.0684	34	0.81	0.2514	68
0.12	0.00269	53	0.47	0.0720	36	0.82	0.2583	69
0.13	0.00326	57	0.48	0.0757	37	0.83	0.2652	69
0.14	0.00389	63	0.49	0.0795	38	0.84	0.2722	70
0.15	0.00456	67	0.50	0.0833	38	0.85	0.2792	70
0.16	0.00527	71	0.51	0.0872	40	0.86	0.2863	71
0.17	0.00608	81	0.52	0.0914	41	0.87	0.2935	72
0.18	0.00700	92	0.53	0.0956	42	0.88	0.3008	73
0.19	0.00804	104	0.54	0.0999	43	0.89	0.3082	74
0.20	0.00918	114	0.55	0.1042	43	0.90	0.3156	74
0.21	0.0104	122	0.56	0.1086	44	0.91	0.3230	74
0.22	0.0117	13	0.57	0.1132	46	0.92	0.3305	75
0.23	0.0130	13	0.58	0.1179	47	0.93	0.3381	76
0.24	0.0144	14	0.59	0.1227	48	0.94	0.3458	77
0.25	0.0158	14	0.60	0.1276	49	0.95	0.3535	77
0.26	0.0173	15	0.61	0.1326	50	0.96	0.3612	77
0.27	0.0189	16	0.62	0.1377	51	0.97	0.3690	78
0.28	0.0207	18	0.63	0.1429	52	0.98	0.3768	78
0.29	0.0226	19	0.64	0.1481	52	0.99	0.3846	78
0.30	0.0246	20	0.65	0.1534	53	1.00	0.3924	78
0.31	0.0267	21	0.66	0.1588	54			
0.32	0.0289	22	0.67	0.1643	55			
0.33	0.0311	22	0.68	0.1699	56			
0.34	0.0333	22	0.69	0.1756	57			

With the use of Table I and a table of areas of segments, the height of the jump may be found by trial.

All functions, such as depth, velocity, and area just above the jump, are known. These quantities may be substituted in Equation 4, and that formula solved for a_2, obtaining an expression of the form,

$$(P_2 - M)\,a_2 + N = 0 \ldots\ldots [16]$$

in which $M = P_1 + 2\,a_1\,h_1$, and $N = 2\,a^2_1\,h_1$, both of which are known. It is then necessary to find values of P_2 from Table I, and of a_2 from any table of areas of segments of circles that will simultaneously satisfy Equation 16. Once the area is known, the height of the jump, or y_2, can be immediately found.

The formulas for standard sections other than that for segments of circles may be solved directly for J by trial. Knowing J, y_2 is found at once, and from it all the other hydraulic elements below the jump.

ENERGY LOSSES IN THE HYDRAULIC JUMP

The energy for constant flow at any section of the conduit is given by the equation:

$$e = y + \frac{V^2}{2\,g} \ldots\ldots [17]$$

The energy lost in the jump is given by the difference in the energies above and below, that is, by

$$i = e_1 - e_2$$

in which i is the eddy loss.

Once the height of the jump is determined, all the hydraulic elements in the section just below the jump become known, and from these the energy loss is readily found.

Each of the foregoing types of conduits has been investigated as to the behavior of a jump within it for identical hydraulic elements above the jump. These elements were arbitrarily given the following values:

Q = 100 cu ft per sec — $e_1 = y_1 + \frac{V^2_1}{2g} = 7.65$ ft

y_1 = 1.425 ft

a_1 = 5.0 sq ft — $r = h_1/y_1 = 4.36$

V_1 = 20.0 ft per sec — $y_2 = J y_1 = 1.425\ J$

Each conduit has at least one parameter, which may be found from the foregoing values. If there are two parameters, one is arbitrarily chosen. For instance, the parameter for the rectangular section is the bed width, $b = a/y$, which is found to be 3.51. For the triangular conduit, the parameter is $k = \tan {}^1/_2\theta$, which is found by substituting the known values in the equation, $a = k\,y^2$. For the trapezoidal conduit there are two parameters, the bed width and the side slopes. Two values for the side slopes were assumed, and for each the bed width and the characteristics of the jump were found.

The process of finding the characteristics of the jump will be illustrated for a trapezoidal section with side slopes of $1^1/_2$ to 1. The area, a_1, equals $b\,y_1 + s\,y^2_1$. Substituting the foregoing values, b is found to equal 1.37 ft. Also, $b + s\,y_1$ equals 3.51. When these values are substituted in Equation 12, it reduces to the form of Equation 13 with numerical values, thus

$$J^4 + 2.65\,J^3 + 3.21\,J^2 - 41.65\,J = 70.5$$

A few trial values will soon show that a value of 3.09 for J will satisfy the equation. Then the following values are computed:

y_2 = 4.41 ft — h_2 = 0.13 ft

a_2 = 35.1 sq ft — $e_2 = y_2 + h_2 = 4.54$ ft

V_2 = 2.85 ft per sec — $i = e_1 - e_2 = 3.11$ ft

For the circular section it is necessary to find a segment having a center depth of 1.425 ft and an area of 5.0 sq ft. This is readily done by trial with the aid of a table of areas of segments. For the case in hand, Z equals 0.25 and D equals 5.70 ft. From Table I,

A DOUBLE JUMP AT OUTLET OF TWIN FLUMES

P_1 is found to be $0.0158\,D^3$, that is, 2.92 cu ft of water. Equation 16 then becomes

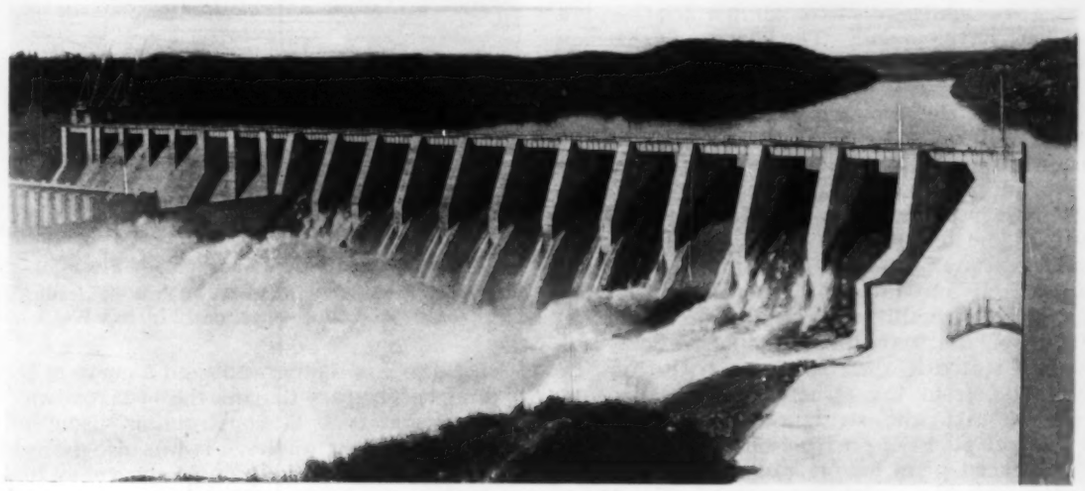
$$(P_2 - 65.12)\,a_2 + 311 = 0$$

With the aid of Table I and a table of areas of segments, D being known, a few trials will show that Z = 0.85. Then y_2 = 4.85; a_2 = 23.1; and P_2 = 51.6.

TABLE II. CHARACTERISTICS OF HYDRAULIC JUMP
In Channels of Given Cross Section with Identical Hydraulic Elements Above the Jump

CONDUIT	PARAMETER	y_2	h_2	e_2	ENERGY LOSS Feet	ENERGY LOSS Per Cent
Rectangular	b = 3.51	5.27	0.45	5.72	1.93	25.2
Triangular	k = 2.46	4.12	0.09	4.21	3.44	45.0
Parabolic	K = 2.94	4.64	0.18	4.82	2.83	37.0
Trapezoidal (s = $1^1/_2$:1)	b = 1.37	4.41	0.13	4.54	3.11	40.6
Trapezoidal (s = 1:1)	b = 2.00	4.62	0.16	4.78	2.87	37.5
Circular	D = 5.70	4.85	0.29	5.14	2.51	31.8

In Table II is given a summary of the heights of jumps and energy losses for each case considered, for identical values of depth, area, and velocity just above the jump.



CHUTE Á CARON DAM OF THE ALCOA POWER COMPANY, ON THE SAGUENAY RIVER, QUEBEC
Discharging 196,000 Cu Ft per Sec; Ogee Spillway at Right Wing Operates Only When Flood Exceeds 340,000 Cu Ft per Sec. Photograph Courtesy of James W. Rickey, M. Am. Soc. C.E.

Highway Bridges for Curved Alignment

Westchester County Park Commission Develops a Unique Type of Structure

By Arthur G. Hayden

Member American Society of Civil Engineers

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ABNORMALITIES result when an attempt is made to design the ordinary type of highway bridge on a heavy grade and sharp curvature, with a deck banked on the outside of the curve to compensate for the overturning effect on moving vehicles. An interesting solution of this problem has been worked out by the Westchester County Park Commission. The design is made up of a series of units, each consisting of a central column from which radiate cantilever brackets to support the floor slab or deck. This design has been utilized for one bridge in the county parkway system and is proposed for another. It has proved to be both architecturally pleasing and economical in cost.

SINCE the types of bridge structures with which engineers are most familiar are not always best adapted to the conditions encountered in practice, new solutions must be found. The rigid-frame bridge, developed and used extensively by the Bronx Parkway Commission and its successor, the Westchester County Park Commission, was so well adapted to the separation of grades between intersecting highways when headroom over the lower highway was restricted, that it has displaced older types of structures to a large extent, in this country as well as abroad, for many uses in addition to the separation of grades. In the construction of the parkway system in Westchester County other special problems have arisen and have been solved in an original manner.

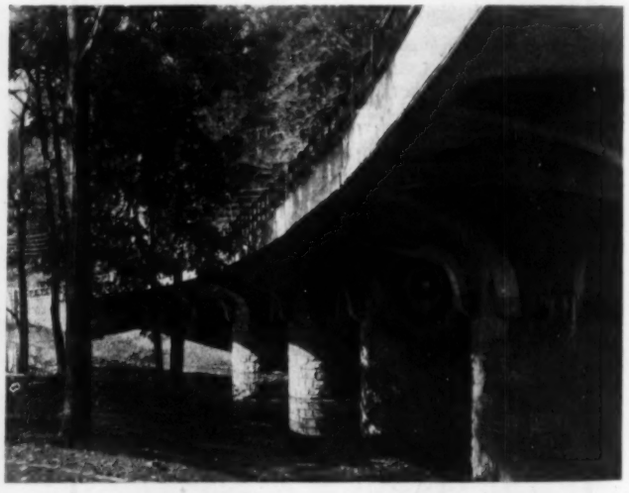
What is believed to be a unique type of structure has been designed to meet a situation for which known types did not appear to be adapted. One such bridge has already been constructed on the Bronx River Parkway at Scarsdale, N.Y., and has been in service over eight years, but improvements are embodied in the more recent design for the Central Westchester Parkway. The conditions leading to the design of the older structure will first be described so that its adaptation to somewhat different conditions at the Central Westchester Parkway crossing may be better understood.

SCARSDALE BRIDGE AN EXAMPLE

In the river valley at Scarsdale the Bronx River Parkway crosses a small lake, surrounding which there is a picturesque park development. The lake is an expansion of the Bronx River, and the parkway, after crossing the lake, runs along the bank of the river. It was necessary to make the crossing on a down grade of 5.5 per cent and on a long curve having a 640-ft radius, necessitating banking of the roadway over the structure 12 in. in a width of 40 ft. The picturesque surroundings demanded a structure of architectural merit. The clearance between the parkway level and the lake was too small to accommodate light open-spandrel arch construction. After several studies of various types of structures had been made, the design shown in one of the accompanying illustrations was adopted as best suited to the conditions, both from the architectural and from the structural point of view. Taken from beneath the structure, the photograph shows the general plan of construction, a series of cylindrical stone-faced piers having radiating brackets to support the bridge floor slab—a very simple design which proved economical as compared with other types.

Each pier with its brackets and floor is a separate unit, but the contiguous floor edges are keyed together so that shear, but not bending moment, can be transmitted from one unit to another. This prevents unequal deflection for unequal loading of adjacent spans. The keys are designed to permit expansion and contraction due to temperature changes. The piers are spaced 40 ft on centers, and the roadway is 40 ft wide between curbs. The proportions for this design as first determined by the designing engineers were very much like those of the proposed Central Westchester Parkway Bridge shown in another illustration, but the consulting architect for the Scarsdale Bridge preferred the heavier piers and shallower brackets used.

The general conditions controlling the design of the Central Westchester Parkway Bridge are similar to those which applied in the case of the Scarsdale Bridge. After crossing the New York Central Railroad tracks, the Central Westchester Parkway passes over the Bronx River and valley, including the Bronx River Parkway drives, as shown in the layout plan, Fig. 1. The crossing



Novel Construction Adopted for Scarsdale Bridge

Piers on 40-Ft Centers; Roadway 40 Ft Wide; Radius of Curvature 640 Ft; and Super-elevation 12 In. in a Width of 40 Ft

is on a grade of 4.8 per cent and a curve of 480-ft radius, making it necessary to bank the 44-ft roadway 20 in.

The usual type of construction, using piers, would require bad skew angles. It was also desirable that the view up and down the picturesque valley be obstructed as little as possible. The design that appeared best adapted to these conditions is shown in the accompanying architect's perspective. The structure is simply a succession of independent units, each consisting of a

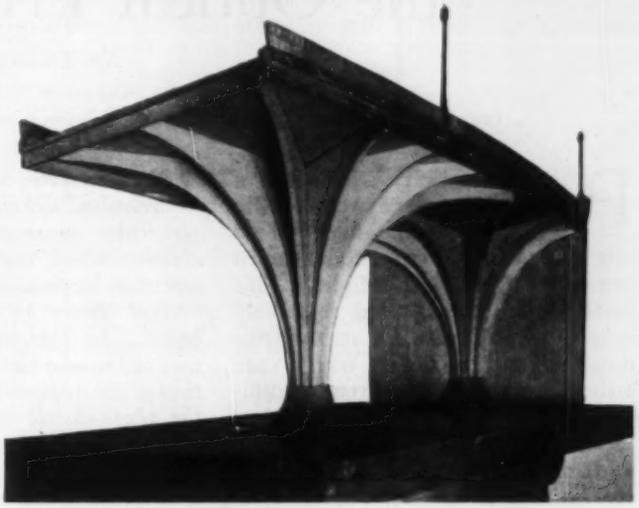
floor slab or deck, supported on cantilever brackets radiating from a central column. The column of each unit is supported on a spread base, which distributes the load over the requisite ground area. The over-all width of the bridge is to be 52 ft, and the distances from center to center of the columns average 70 ft.

Several problems of detail were encountered in the design of the structure, the most important being the make-up of the central columns for steel construction. Several designs were made and compared as to cost and ease of erection of the structure as a whole. The most favorable type was a 60-in. steel pipe resting on a steel plate in the concrete base pedestal. The brackets are all tied together over the top of the column by a steel plate. The vertical bracket loads are carried directly to the footing through vertical angles, which are merely stitch-bolted to the pipe column.

The pipe column is designed to carry bending due to eccentricity of live loads. This bending is resisted at the bottom by adequate embedment in the concrete pedestal, and special circumferential reinforcement in the pedestal prevents its splitting under the lever action of the pipe column. The maximum average soil pressure on the pedestal base is a little over $1^1/_4$ tons per sq ft, and the maximum extreme soil pressure at the circumference of the pedestal under eccentrically applied live loading will be $2^1/_2$ tons per sq ft.

PROPOSED CENTRAL WESTCHESTER PARKWAY BRIDGE
Architect's Perspective of Design with Structural Steel Brackets

From the photograph of a wooden model of one span, an idea may be gained of the appearance of such a bridge built with solid concrete or steel plate-and-angle brackets. The steel latticed brackets are favored because of their lightness of construction, but it is an open question which type of construction would be most pleasing in general effect. Although the general design appears unusual, the structural idea is simple and direct, and preliminary estimates indicate that it will be economical.



A PROPOSED DESIGN SHOWING APPEARANCE OF SOLID CONCRETE BRACKETS
Wooden Model by James Albro

ACKNOWLEDGMENTS

The architectural design was prepared by Clinton F. Loyd under the direction of Gilmore D. Clarke, landscape architect on the staff of the Westchester County Park Commission, and the structural design is being made by Richard M. Hodges under my general direction. Jay Downer is Chief Engineer of the Westchester County Park Commission and Leslie G. Holleran is Deputy Chief Engineer, both Members Am. Soc. C.E.

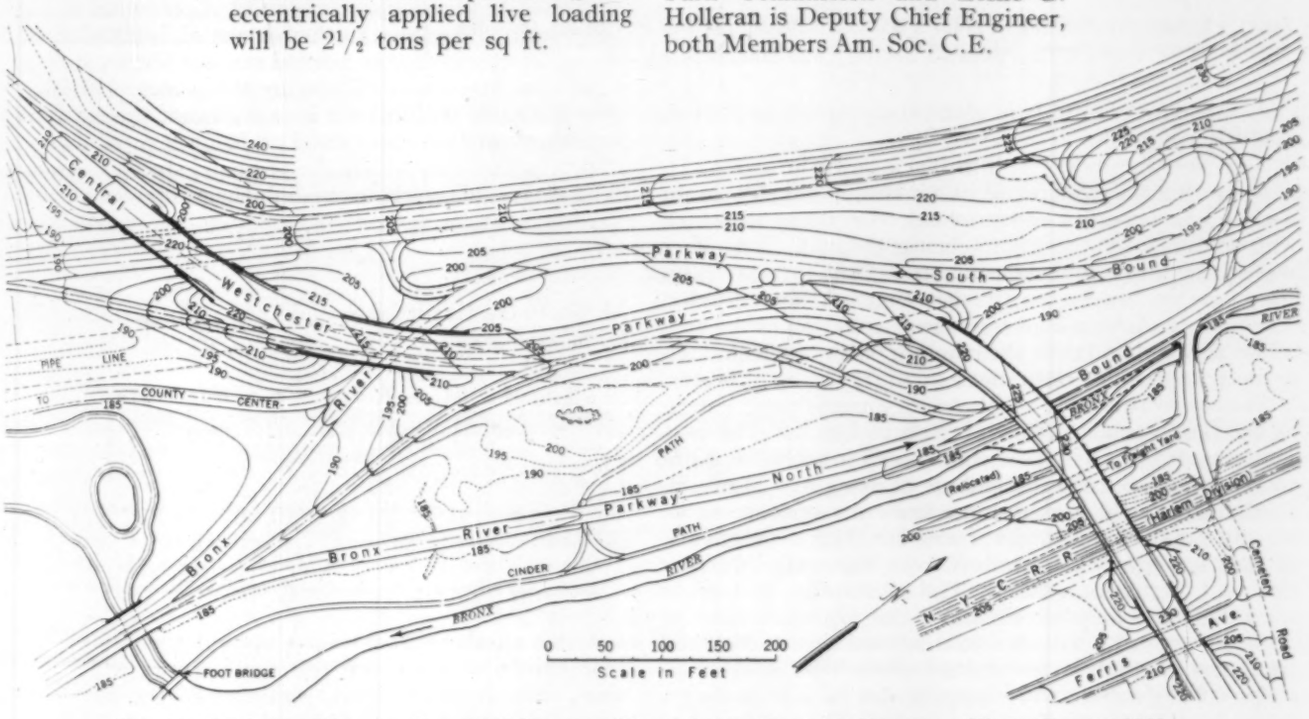


FIG. 1. SITUATION MAP FOR PROPOSED CENTRAL WESTCHESTER PARKWAY BRIDGE AT WHITE PLAINS, N.Y.

Photo-Elasticity—A Short Explanation of the Optical Principles Involved

By THOMAS H. EVANS

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IN connection with the many current technical articles on the analysis of particular stress problems by the photo-elastic method, engineers will find of interest an explanation of the elementary optical theory on which the method is based. In this article by Mr. Evans will be found a clear, elementary description of the apparatus used in analysis by the photo-elastic method. His explanation makes it easy to interpret the meaning of the many photo-elastic pictures now found in engineering literature.

PHOTO-elasticity furnishes a method of stress analysis based on the effect of a stressed, transparent model on a beam of polarized light. This method has been found relatively quick and accurate for many two-dimensional problems which are either impossible, or extremely difficult of solution by the mathematical method. And it is a very simple and rapid procedure where the chief aim is to locate areas of stress concentration in a member.

In consideration of the current revival of such a valuable method of analysis, an outline of the elements upon which the photo-elastic method is based is timely.

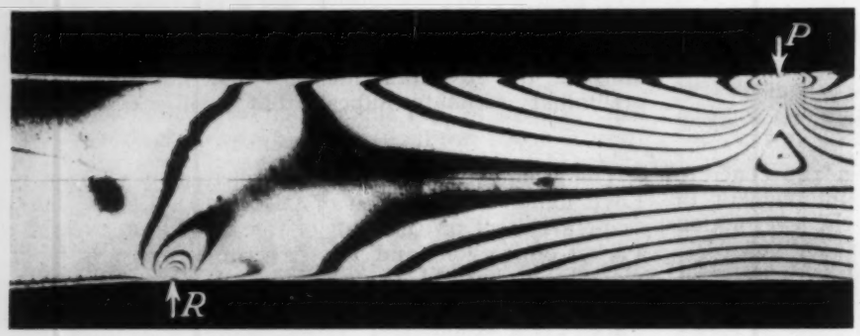

FIG. 1. COMPOSITE PICTURE OF HALF A BAKELITE BEAM, AS SEEN IN THE POLARISCOPE
Under a Concentrated Central Load Simply Supported

Only the bare essentials of elementary optics and stress analysis are given.

Consider the photograph in black and white (Fig. 1), made with a polariscope, of half a simple bakelite beam with a concentrated central load, the stress in which was analyzed by the photo-elastic method. The lines are contours of equal shearing stress, but in most cases they have no bearing on the directions of those stresses. The picture shows a high stress concentration in the area under the load and over the reaction, both of which, incidentally, are neglected in the elementary theory of beams.

The apparatus used in photo-elastic analyses, known as the polariscope, is shown set up in Fig. 2. The purpose here is only to explain the theory involved and not to attempt to outline the technic of using it. Many investigators at the present time favor a monochromatic, or single wave-length light source for their experiments. A good green ray is obtained from the mercury-vapor arc with the use of a filter. The assumption of a single wave length somewhat simplifies the explanation of all optical theory and will therefore be used here. It should be borne in mind, however, that white light, which consists of innumerable wave lengths, can be and is used a great deal in photo-elastic analysis. The colored pictures often seen can be obtained only by its use.

In elementary textbooks on mechanics it is demonstrated that for an elastic material subjected to a system of plane stresses there are two mutually perpendicular planes through any element which have only normal stresses acting on them. The stress system acting on any element, A, of the body shown in Fig. 3 (a), is given in Fig. 3 (b). For this particular point the two perpendicular planes just mentioned have a certain orientation, the angle θ in Fig. 3 (b) and (c) and the angle $\theta + 90$ deg (not shown). On any other planes through that point there may be shearing stresses in addition to a normal stress, as shown in Fig. 3 (d). Only two-dimensional stress systems can be considered in photo-elasticity.

These two particular normal stresses which occur on mutually perpendicular planes that are free from any shear, are known as principal stresses on an element. They will hereafter be designated as P and Q, and assumed as tensions. Now if an element in a material is so oriented that it has only normal stresses on its boundaries, these are the principal stresses. In Fig. 3 (a), an element at A, oriented at θ degrees to the horizontal, is acted on only by the normal stresses shown in Fig. 3 (e). These are the principal stresses.

In the same textbooks it is proved that the maximum possible shear that can exist at a point acts on planes at 45 deg to those on which the principal stresses act, and has a magnitude equal to one-half their difference, that is,

$$S_{max} = \frac{P - Q}{2} \quad . \quad . \quad . \quad . \quad . \quad . \quad [1]$$

All photo-elastic analysis, and theory, involves so-called principal stresses.

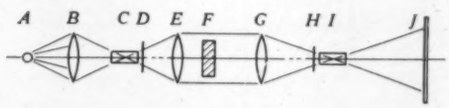

FIG. 2. SCHEMATIC LAYOUT OF A POLARISCOPE
(A) Light Source, (B) Lens, (C) Nicol Prism (Polarizer), (D) Quarter Plate, (E) Lens, (F) Model, (G) Lens, (H) Quarter Plate, (I) Nicol Prism (Analyzer), and (J) Screen or Camera

Although there exist today several theories of light, that known as the transverse wave theory will be used here, since it satisfactorily explains all the phenomena involved in photo-elasticity and is well known to those

who have studied elementary physics. Basically, it assumes that light exists because of transverse vibrations of the ether particles, which may be thought of as having the same motion as do the particles of a stretched wire when struck perpendicular to its length. By using this rather simple, and satisfactory, convention of transverse waves, the theory of polarized light is more easily understood. The two basic phenomena upon which all photo-elastic theory hinges are those of polarization and double refraction of light.

Common light is made up of vibrations in an infinite number of planes containing the line of propagation of the ray. That is, there is no one single plane in which the particles vibrate as they do in the wire analogy previously mentioned.

For the purposes of photo-elasticity it is necessary to give the vibrations some definite orientation, which is called polarizing. There are three possibilities here; plane, circular, and elliptical polarization, but the former two are simply special cases of the latter. In the polariscope, use is made principally of plane polarization, although it is desirable at times to use circular. Since an understanding of elliptical and circular polarization is not absolutely essential to an interpretation of photo-elastic pictures and since the theory may be found in any standard textbook on college physics, it will not be included here.

Plane polarization means that the ether particles are made to vibrate in only one definite plane. If an end view of the vibrations were possible, the particles would be seen to move back and forth in simple harmonic motion on one straight line only, the length of which is equal to twice the amplitude of vibration. The amplitude and direction of vibration may be represented by a vector, which may be broken down into components, if desired, just as may be done with any vector quantity.

The second basic phenomenon of photo-elasticity is that of double refraction. The term refraction means the change in direction imposed upon a light ray due to a change in speed, as it passes from one medium into another, such as from air to glass. It is an experimental fact that certain transparent crystals, such as Iceland spar ($CaCO_3$), have the peculiar property of breaking a beam of light into two rays, each of which becomes plane polarized but vibrates at 90 deg to the other.

Such crystals are permanently non-isotropic, that is, they have different properties in different directions, and hence the passage of light through such a medium is favored in one particular plane over all others. The former is known as the principal plane. Since ordinary light vibrations may be represented by two mutually

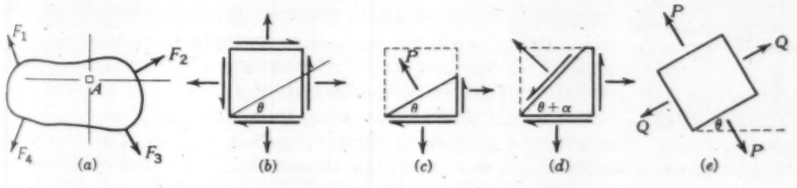

FIG. 3. PRINCIPAL AND PRIMARY STRESSES ACTING ON AN ELEMENT

perpendicular vectors, that parallel to this principal plane will have greater speed of propagation than the one perpendicular to it. Hence the incident beam is broken down into two plane polarized rays which have slightly divergent paths, and different speeds, through the crystal. This is known as the phenomenon of double refraction and the rays are given the optical names "ordinary" and "extraordinary."

This phenomenon is utilized in the construction of a device that will plane polarize a ray of ordinary light. The oldest and most common such device is known as a Nicol prism, which is made up as shown in Fig. 4. The crystals of Iceland spar are so cut and cemented together that one ray is refracted and strikes the sides before it emerges from the opposite end. Since the sides are

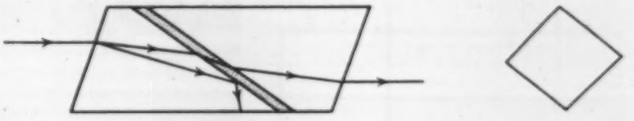

FIG. 4. CONSTRUCTION OF A NICOL PRISM
Two Crystals of Iceland Spar Cemented with Canada Balsam

blackened, this ray is absorbed and lost, and only one ray of plane polarized light emerges. There are several polarizers on the market, but they are all much the same in principle as the Nicol prism.

Certain materials have equivalent properties—such as the conduction of heat, conduction of light, and elastic behavior—in all directions. It has been proved by experiment that these so-called isotropic materials, including glass, celluloid, and bakelite, become temporarily non-isotropic when subjected to stress and cause double refraction of light, as do the crystals of Iceland spar.

Every element in a body subjected to plane stress has acting on it two principal stresses, P and Q, at right angles to each other. It so happens that these directions of principal stress at any point determine the two planes into which the beam of incident light becomes polarized when temporary double refraction takes place. Thus in Fig. 5, if the directions of the principal stress at points A and B are as shown when the load, F, is applied, and a

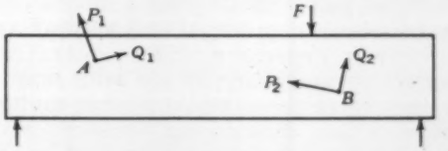

FIG. 5. DIRECTION OF PRINCIPAL STRESSES AT TWO POINTS IN THE LOADED MODEL OF A BEAM

ray of ordinary light strikes each point perpendicular to the plane of the paper, this ray will be broken down into two rays, each of which is plane polarized and vibrates in the directions of the principal stresses. These rays correspond to the ordinary and extraordinary rays mentioned in connection with certain crystals.

If, instead of natural light, a beam that has already been plane polarized is used, this latter will be broken down at each point into two component plane polarized rays. The planes of vibration will have the directions of the principal stresses at that point.

Because one ray travels faster than the other through the medium, one emerges before the other and vibrates out of phase with it. That is, at the same point and time, one wave train lags behind the other. This particular phase difference will continue to exist until the rays pass into another medium. These two rays are simply components, as brought out above, and can therefore be resolved into their resultant at any point along the path. Now if two mutually perpendicular, simple

harmonic vibrations are out of phase by any number of whole wave lengths, the emerging resultant vibration is unchanged in kind or orientation with respect to that which entered the medium. If, however, there is a phase difference of an odd number of half waves, then the plane of the original polarization will have been rotated through 90 deg. In other words, if the plane of vibration was originally vertical, it would be horizontal after passing through a crystal which set up a phase difference of some odd number of half waves between the ordinary and extraordinary rays. There are other possibilities for other retardation, but these two will suffice for the present.

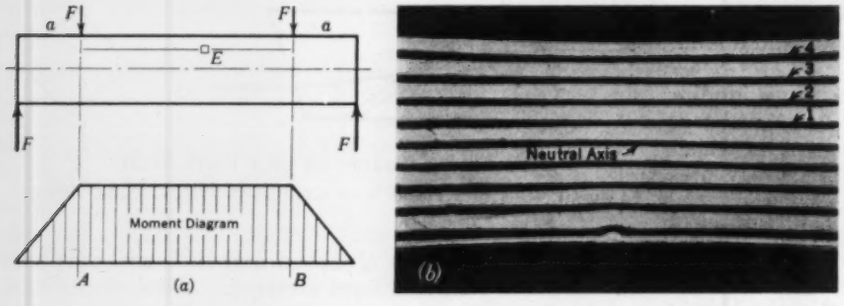

FIG. 6. METHOD OF LOADING AND PHOTO-ELASTIC PICTURE OF MODEL IN PURE BENDING

The stressed transparent medium—celluloid or bakelite—therefore sets up at every point a phase difference, or retardation, between the two rays whose planes of polarization are mutually perpendicular. Unlike the unstressed crystal, however, the speed of a ray along either of the two planes depends upon the corresponding principal stress. Hence the phase difference set up is a direct function of the difference between the principal stresses at the point under consideration. Since the difference between the principal stresses may vary from point to point in a specimen, it follows that the phase differences set up may vary in the same way across a material—anywhere from zero to some multiple of a whole wave. This relationship between retardation and principal stress difference is expressed by an empirical law known as the stress-optic law, which is

$$R = c(P - Q)t \quad . \quad . \quad . \quad . \quad . \quad . \quad [2]$$

in which R is the retardation in wave lengths, c is an optical constant of the material, and t is the thickness. If the material is of uniform thickness, c and t may be combined into one constant.

ELEMENTS OF THE POLARISCOPE

With all this optical theory in mind, the polariscope as a whole can be studied. The complete set-up is shown in Fig. 2. The only function of the various lenses is to cause the light beam to spread or converge where necessary. Consider first the two Nicol prisms alone. These are constructed exactly alike; therefore, if their longitudinal axes are placed in line, and if the corresponding sides are in the same planes, then the polarized ray that gets through the first, called the polarizer, will pass unchanged through the second, called the analyzer. If, however, the second is rotated through 90 deg—known as "crossing the Nicols"—then no light will get through the second. Now consider the model in the system. The first Nicol prism polarizes the light ray, which is then spread so that it encompasses the whole model. At each point in the stressed model the element of light passing through that point is broken down into two mutually perpendicular, plane vibrations, which are also out of phase in direct proportion to the difference between principal stresses at that point. Hence there are many phase differences throughout the model, but only those which are multiples of half waves are of interest in this connection. Whenever a difference of an odd number of half waves exists, the plane of original polarization has been rotated through 90 deg; and it has gone through unchanged for any number of whole waves difference, since this in reality amounts to no phase difference.

If the Nicol prisms are now crossed, and the image viewed on a screen, maximum brilliance will occur at points where there is a difference of an odd number of half waves, because the plane of polarization has been rotated 90 deg, and the analyzer was set to pick up such plane vibrations. Maximum darkness, or black fringes, will show up at points of even half-wave diffeences. On the screen there will be a gradual change from one maximum to the next. If there are connected points of equal stress difference, then a continuous band will appear, such as those shown in the photographs. Since the stress function is continuous, a picture of the specimen will always show a series of alternate black and white bands, the pattern's complexity varying as the loading. The term "fringe" is usually used for a black band only. The white bands seen on photographs have, of course, the color of the light source when projected on a screen.

There are two special cases of principal stress values which may be of interest. One occurs when P and Q have the same value, when from Equation 2 it is seen that the retardation is zero. The same thing is true if P and Q are both equal to zero. Both these cases may occur in the same model, and such points show up black on the picture, since the original ray suffers no change. These spots can be distinguished from the fringes, and it is also possible to differentiate between the two special cases.

MEANING OF BLACK AND WHITE BANDS

Thus the black and white bands in Fig. 1 are contours of equal stress differences. Since $\frac{P - Q}{2}$ equals the maximum possible shear on an element, the bands may also be taken as contours of equal maximum shear. An important point, however, is that there is nothing in the picture which necessarily has anything to do with stress directions, because these were in no way related to the phase differences set up. The picture is important because it shows graphically the stress distribution for the particular system of loading used.

Stress concentrations occur wherever there is a marked crowding together and curving of lines, such as under a load, where the crowding shows that the stress values are changing rapidly. If an investigator is interested simply in finding concentrations, it is only necessary to build a model of the cross section and view it in the polariscope while it is subjected to the proper stress system. If it is desired to reduce any concentrations, it may be done by making such changes in the model as will show less crowding and curvature of the fringes. When it is required to find the values of the maximum shearing stress represented by the fringes, however, the picture means little unless subsequent photo-elastic work is done.

One common method of determining fringe values is

to set up a beam of uniform thickness, in such a way as to get pure bending over a part, such as AB in Fig. 6 (*a*). A photograph similar to that in Fig. 6 (*b*) is obtained, in which it will be noted that the fringes are all straight and parallel to the edges, the number of course depending on the moment imposed. By using the well-known flexure formula,

$$\frac{My}{I} = S. \quad \ldots\ldots\ldots \quad [3]$$

the stress value of any fringe in the picture can be found. The only stresses that occur in a beam in pure bending are parallel to the edges, and are either direct compressions or tensions. If an element, such as E in Fig. 6 (*a*), were shown as a free body, it would have acting on it only compression in one direction, since there is no normal stress on the other two sides. Therefore this is one principal stress, the other being zero, and its value, from the flexure formula, is the difference in the principal stresses at that point and at every point on the same fiber within the part AB. Thus the fringes, which represent equal differences of principal stress, are parallel to the edges in this case of pure bending. The maximum shearing-stress value is therefore equal in magnitude to half this direct stress. For the case of pure bending there are corresponding fringes, with equal values, on each side of the neutral axis.

FRINGES APPEAR IN SUCCESSION AS LOAD INCREASES

Of course all fringes look exactly alike, but they come into the picture in succession as the load is increased. In the case under consideration more and more crowd into the picture as the load is put on the model. The order of a fringe, that is, its relative stress value, can be determined by counting from the fringe which was observed to come into the picture first. Thus the shear values of orders in the first fringe photograph, Fig. 1, can be fixed by determining that of corresponding orders in Fig. 6 (*b*) and taking account of any difference in thickness in the two models. Subsequent orders should represent whole multiples of the stress in the first.

Up to this point in the analysis both the distribution and values of the maximum shearing stress have been made determinable. It is also possible to find the values of the two principal stresses at each point, but it requires a great deal more very elaborate analysis, most of which is mechanical or graphical. The directions of the principal stresses can also be found with the polariscope, and it is necessary to do this when it is desired to carry out a complete analysis of the principal stress values. If a plane polarized ray strikes a stressed model parallel to the direction of some principal stress, it can have no component parallel to the other principal stress and will therefore suffer no retardation. Since principal stresses occur in the model at all orientations, there will usually be a black line or two on the model image showing a locus of points whose principal stress directions are parallel to the principal axes in the crossed Nicol prisms.

HOW TO OBTAIN DIRECTIONS OF PRINCIPAL STRESSES

If the Nicol prisms remain crossed, but are rotated about their long axes, other lines can be obtained across the model for other settings of the prisms. By tracing these out on the model image for 5-deg rotations of the prisms, from zero to 90 deg, a series of contours is obtained, from which can be found the directions of the principal stresses by graphical methods. At any free boundary, of course, there can be only one principal stress, and it is parallel to the boundary. It should be pointed out that these black direction contours, or isoclinics, confuse the fringe pattern by crossing it, but do not change the stress lines at all.

When it is desirable to see only the fringe pattern, unconfused by the isoclinics, the latter are eliminated by an optical device known as a quarter-wave plate. This is made of a doubly refractive material of such thickness that it is capable of giving circularly polarized light. In Fig. 2 it is seen that two quarter-wave plates are used when it is desired to eliminate the isoclinics. The circularly polarized light from the first quarter plate, D, eliminates all effects that the stress directions may have had on plane polarized light. The second quarter plate, H, nullifies only the effect the first had on the light, but not that caused by the model itself. The quarter plates therefore have absolutely no effect on the stress pattern other than to extinguish the isoclinics.

All initial internal stresses, such as those caused by forming the model, must be removed to eliminate the possibility of confusing them with induced stresses. The extent of any initial stress can be observed with the polariscope when the specimen is under no load, and most of it can be removed by careful annealing.

The theory advanced applies to white light as well as to that of a single wave length, although the latter assumption serves to clarify the explanation. Several of the more important investigators in the field today use white light exclusively in preference to the so-called fringe method of monochromatic light. The major difference in results is that there are no black fringes on the screen when a white source is used, but instead a similar pattern made up of yellow, red, and green bands (in bakelite), which have the same meaning as the fringes.

WHITE LIGHT NOT EASY TO PHOTOGRAPH

One disadvantage of the white-light method is that it cannot be photographed as conveniently as can the fringes, which give a simple black and white picture. The appearance of colors on the screen can be explained on the same basis as were the dark and light bands. Interference with the latter gave light, or no light; with white light it gives bands of complementary colors.

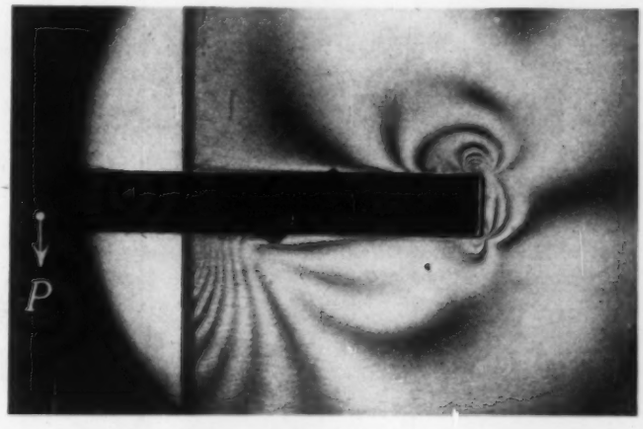

FIG. 7. STRESS PATTERN IN A BAKELITE PLATE DUE TO CANTILEVER ACTION OF THE STEEL BAR

A black and white picture of the stress contours (isochromatics) set up in a bakelite plate when loaded by means of a steel rod under cantilever action is shown in Fig. 7. It should be noted that there is heavy stress concentration at the main bearing surfaces and the sharp corners. Some of this latter might be removed by rounding the corners.

Some Shear Phenomena in a Loaded Soil Mass

Experimental Evidence That Loaded Soils Possess Elastic Characteristics

By DIMITRI P. KRYNINE

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DURING recent years the mechanical action of soils has received increased attention from investigators, notably Charles Terzaghi, M. Am. Soc. C.E., and the Society's Committee on Earths and Foundations, whose report appeared in PROCEEDINGS *for May 1933. A member of the committee, Professor Krynine, has been conducting recent investigations on laboratory samples of soils to determine their action under load. Following the methods of study of the elastic properties of metals, as described by A. Nádai, Professor Krynine has tested thin soil layers by applying loads at their edges. He observed the formation of a circular or elliptical crack in the sand, corresponding to the isochromatic lines seen in photo-elastic pictures of a celluloid sheet loaded at the edge. In addition, slip lines similar to those found by other investigators in loaded metal and bakelite plates appeared on the surface of the soil layers. By analogy, the experiments indicate the elastic behavior of soils under heavy pressure.*

WRITERS of articles on soil mechanics have dealt with two classes of soils: (1) those below the ground-water level, principally clays; and (2) those above that level, principally dry and moist sands. Theoretical explanations of the behavior of clays below the ground-water level, which have all their voids filled with moisture, have been brilliantly developed by Dr. Terzaghi. Squeezing of moisture from the voids of such a soil mass when loaded is the principal factor controlling the action of the soil. Moist soil masses above the ground-water level seem to behave differently. Obviously, there is also some squeezing of moisture from the pores of such a soil mass when loaded, but as a rule its action seems to be governed by the laws applying to other solid bodies.

Some time ago an interesting book on plasticity appeared. It was written by Dr. A. Nádai, and deals with the mechanics of the plastic state of matter, principally with the mechanics of metals. I have repeated some of the experiments described in that book using not metals but moist soils as the experimental material. The results were strikingly similar to those secured with metals, as described by Dr. Nádai.

BULGING OF SOILS INVESTIGATED

When a load acts on the surface of a body it provokes within it stresses acting in three dimensions. In order to simplify the problem, a thin plate, of metal or celluloid for instance, may be loaded and its behavior observed. Thus only two instead of three principal stresses are considered. In other words, for a space problem a plane problem is substituted. A similar procedure is used in photo-elastic research. In my experiments thin layers of soil were used instead of thin metallic plates.

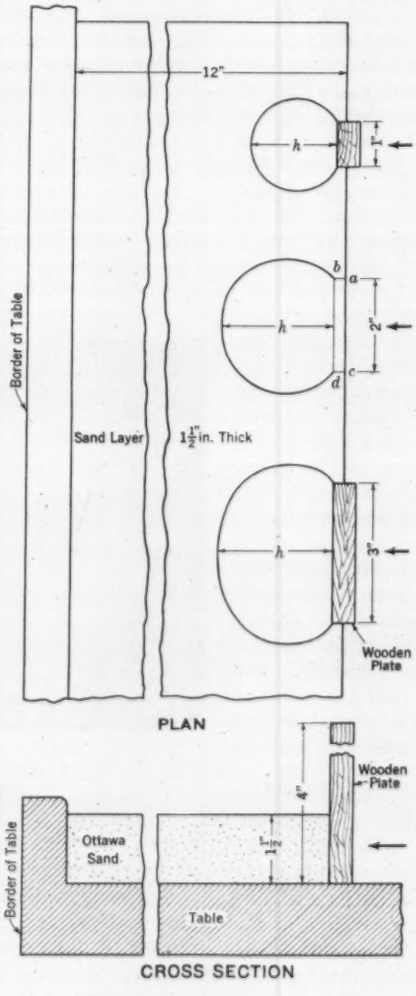


FIG. 1. HORIZONTAL SAND LAYER BULGED AND CRACKED UNDER HORIZONTAL PRESSURE EXERTED BY WOODEN PLATES

According to Dr. Nádai, in some experiments with mild steel a "punch" acting in the plane of the sheet was pressed against the edge of the test piece. Afterwards the steel was reheated until the grains recrystallized. Thus a surface bounded by a line very similar to an isochromatic line in a photo-elastic test was revealed. As in my tests, such isochromatic lines were produced by the simple method to be described.

A compacted layer of moist standard Ottawa sand was placed horizontally on a laboratory table, the vertical border of the table serving as a buttress for one edge of the layer, while the other three edges remained free and stood vertically (Fig. 1). Tests were made on layers of three thicknesses; 1 in., $1^1/_2$ in., and 2 in. Five wooden plates, 4 in. high and $^1/_2$ in. thick, were used. The widths of the plates were 1 in., 2 in., 3 in., 4 in., and 5 in., respectively. A glass plate 10 in. wide was also used for the same purpose. Both the wooden and the glass plates were carefully but strongly pressed by hand against the edge of the layer until bulging occurred, followed by cracking along a curve. As may be seen in Fig. 1, which illustrates the results of the test on the layer $1^1/_2$ in. thick, the wooden plates 1 in. and 2 in. wide caused circular cracks similar to isochromatic lines, and a wider plate gave an ellipse-like crack. In many experiments, not represented graphically here, the minor axis, h (Fig. 1), of the elliptical shapes was independent of the width of the plate and was practically constant for a layer of given thickness. In the case of the circular isochromatic lines, the diameter, h (Fig. 1), varied with the width of the plate through which the pressure was applied.

Similar results were secured when glass pressure plates were used. The result obtained by pressing a wooden

plate 2 in. wide against a layer of moist Ottawa sand 1 in. thick is illustrated in Fig. 2. In this the isochromatic line is emphasized by a scratch in the sand following the crack.

Thus isochromatic lines were obtained only under narrow punches and not under wider ones. Similarly, Dr. Nádai found that if a thin sheet of steel is pressed by a wide punch, it also bulges in the form of an ellipse.

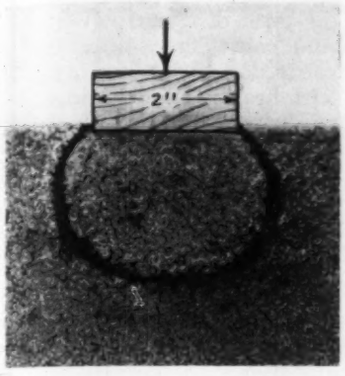


FIG. 2. BULGE PRODUCED IN A LAYER OF MOIST OTTAWA SAND ONE INCH THICK BY PRESSING AGAINST ITS EDGE A WOODEN PLATE TWO INCHES WIDE

FORMATION OF SLIP LINES AT YIELD POINT

Metals are elastic bodies; therefore, if a perfectly rigid punch is pressed against the edge of a metallic test piece, yielding starts at the sharp edges of the punch. The "slip lines" for such a case, as determined by Mesmer in Germany, are shown in Fig. 3. As may be seen, these slip lines, or trajectories of principal shearing stress, represent two systems of curves intersecting at angles of 90 deg. Similar curves on bakelite plates were obtained by A. H. Beyer, M. Am. Soc. C.E., and A. G. Solakian, Assoc. M. Am. Soc. C.E., when conducting their photoelasticity research work for the Society's Committee on Earths and Foundations, of which Lazarus White, M. Am. Soc. C.E., is chairman. Comprehensive information along these lines may be found in the report of the committee, in PROCEEDINGS for May 1933.

In my experiments with soils, slip lines were obtained by pressing a metallic punch, somewhat less than an inch square, against the edge of a horizontal soil layer about an inch thick. When thus pressed, a soil mass did not at first reveal elastic properties, as metals do, and the punch had to be pressed in considerably before the soil would act similar to an elastic body. Three kinds of soils were used, as shown in Fig. 4: (*a*) very fine New Haven sand (only 6.5 per cent retained on sieve No. 100), (*b*) silty fine sand from the shore of the Connecticut river in East Hartford (grains smaller than 0.3 mm with 11 per cent of colloids in the 15 min hydrometer test); (*c*) North Haven clay with a plasticity index of about 15. The moisture content of all these soils before loading was somewhat less than their respective liquid limit.

The slip lines for soils *a*, *b*, and *c* are illustrated in Fig. 4. The circle in Fig. 4 (*c*) was drawn on the surface of the clay layer by hand to outline the boundary of the bulged zone. The slip lines indicate the general behavior, under a heavy load, of the soils studied. Their action appears to be analogous to that of an elastic body. The two systems of shearing trajectories are clearly visible but the angle of their intersection, which should be equal to 90 deg, is actually considerably smaller.

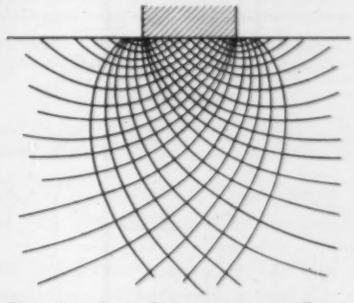


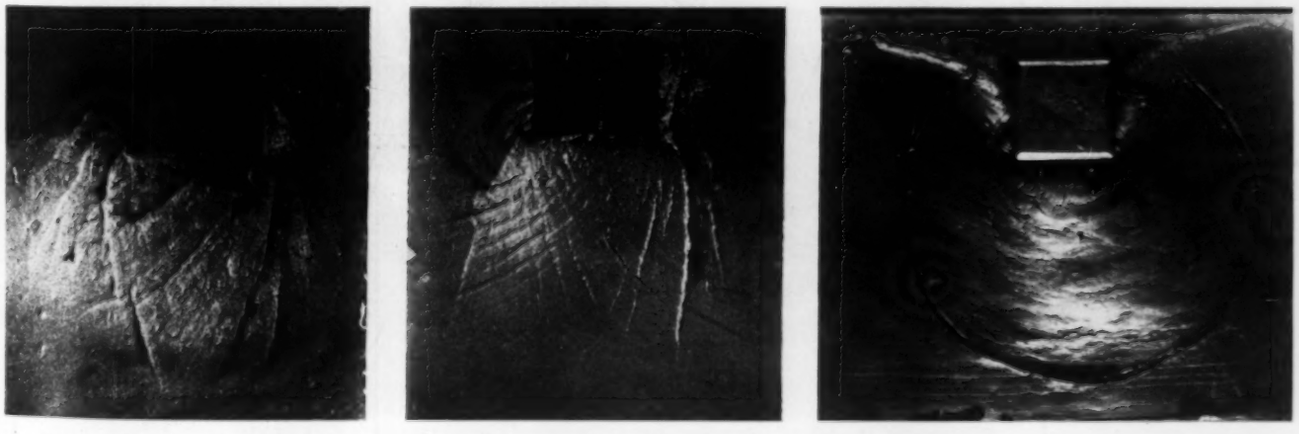
FIG. 3. SLIP LINES FOR A CONCENTRATED CONSTANT PRESSURE ACTING ALONG THE EDGE OF A METAL PLATE

Determined by G. Mesmer; Drawing from *Plasticity*, by A. Nádai

In one of my recent field experiments, the existence of a practically semicircular disturbed zone under a loaded platform was established. Referring to Fig. 1, it should be stated that all the material from the area *abcd* is to be found within the round bulged zone. The bulged zones in Figs. 1 and 2 could form because there was not sufficient load upon them to prevent bulging. If restricted, those zones would have remained in their original position and would have been compressed. If the experiment is made on moist sand placed between two heavy glass sheets, the circles and ellipses do not appear, but nevertheless there is a zone of compression, and when the pressure exceeds a certain limit, the bulge may be seen at the edge of the layer, close to the plate through which the pressure is applied.

DISTURBED ZONE UNDER A BUILDING

An identical condition probably exists under a building. A highly compressed zone, *B* (Fig. 5) develops under a building, *A*, but this obviously does not exclude the possibility that lesser compressive stresses will develop in the rest of the soil, *C*. If the load *A* is steadily increased, stresses acting on the boundary line, *D* of the zone *B*, may exceed the corresponding resistance. In such a case, bonds connecting that zone with the rest of the soil may fail by shear.



(*a*) Very Fine New Haven Sand (*b*) Silty Fine Sand from East Hartford (*c*) North Haven Clay

FIG. 4. FORMATION OF SLIP LINES IN LAYERS OF MOIST SOIL UNDER PRESSURE

Some indications of the possibility of the existence of a soil zone moving together with the building may be found in the well known book in German by H. Krey on soil resistance. As early as 1929, M. Pigoulevsky in Russia proved experimentally the existence of such a zone bounded by a somewhat deformed semicircle.

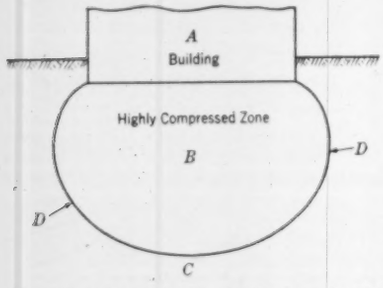


FIG. 5. HIGHLY COMPRESSED ZONE UNDER A HYPOTHETICAL BUILDING
Zone B May Substantially Change Its Physical Characteristics and Act Like a Foreign Body in the Rest of the Soil

In making his interesting sand-cutting experiments in Germany, Johannes Rathje employed a large box filled with moist sand having grains from $^1/_2$ to 2 mm in size. A carriage supporting the recording apparatus and a cutter blade, which moved within the soil mass, could be towed over the box. The cutter had a thickness of 15 mm and a width of 60 mm measured in the direction of its motion. As the cutter moved, a sand wedge was observed to form in front of it. A trough in the path of the cutter was then excavated in the tamped sand. The cutter was placed vertically in this trough and packed all around with colored sand. When the cutter was moved forward through the zone of colored sand until it had progressed beyond it, there was a clearly seen wedge of colored sand carried in front of the cutter. The horizontal section of the wedge was a symmetrical curve approximating a semicircle. The experiments conducted by L. Klein in Germany furnished similar results.

All these analogies suggest that under a narrow building there exists a disturbed zone which has a nearly semicircular cross section and moves downward with the building. While nothing definite can be said regarding wider buildings, it is probable that the disturbed zone under them is ellipse-like or elongated, as shown in Fig. 1.

ELASTIC BEHAVIOR OF A SOIL

An excessive load in the form of a narrow structure (similar to a punch) should cause, first, heavy settlement, and afterwards destruction of the soil mass by shear. Such is the case with heavily loaded bridge piers or grain elevators. Some engineering authors, for instance H. Krey, visualized the soil action under a pier

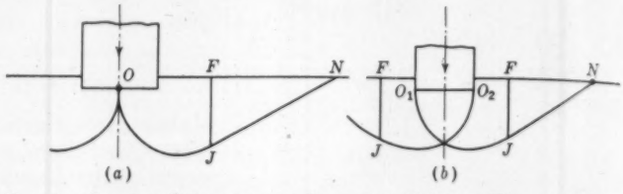
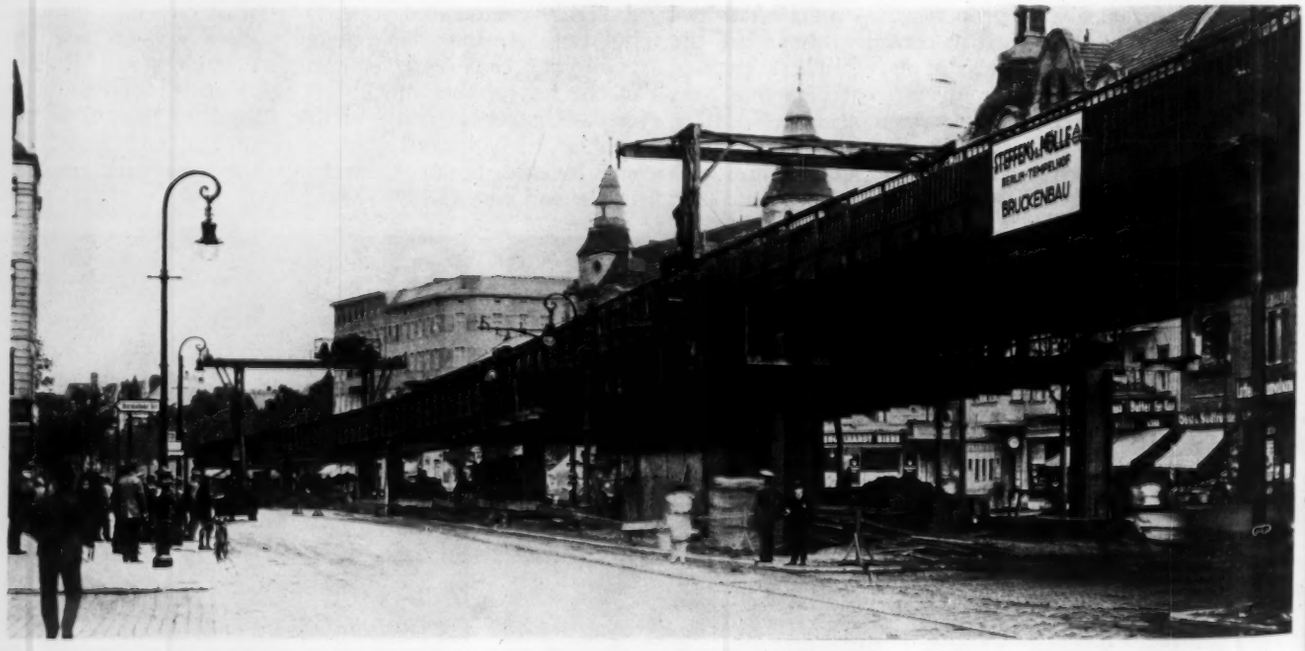


FIG. 6. SLIDING SURFACES AND PRISMS OF PASSIVE THRUST UNDER A BRIDGE PIER, ACCORDING TO H. KREY

such as that represented in Fig. 6 as taking place in the following manner. Curved sliding surfaces would form, either OJ, starting at the center of the base of the pier, or O_1J and O_2J, starting at its edges. Simultaneously prisms of passive thrust, FJN, bounded by a plane, JN, would be pushed out. This opinion seems to be close to the truth if shearing surfaces be substituted for the lines OJN, O_1JN, and O_2JN.

The experiments described make it appear that, as a rule, a recently loaded natural soil mass does not act at once as a perfectly elastic body although it may acquire elastic properties under pressure. In terms of a building, this means that during and immediately after construction the soil is not yet fully elastic but it may become so, at least locally, toward the end of the period of settlement.



A BERLIN BRIDGE CONSISTING OF A GIRDER CONCENTRICALLY CURVED
Two Other Structures of This Type Have Been Erected in the City, on One of Which Welding Was Employed for Both Fabrication and Erection. Photograph Courtesy of Dr.-Ing. Harry Gottfeldt, Berlin, Germany

ENGINEERS' NOTEBOOK

From everyday experience engineers gather a store of knowledge on which they depend for growth as individuals and as a profession. This department, designed to contain practical or ingenious suggestions from engineers both young and old, should prove helpful in the solution of many troublesome problems.

Reinforced Concrete Chimneys

By JOSEPH FERTIK

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FORMERLY STRUCTURAL ENGINEER, ELECTRIC BOND AND SHARE COMPANY, NEW YORK, N.Y.

IN modern practice the engineer is often called upon to design a shell with openings in its sides, for instance, a chimney with one or more flue openings. The chimney is basically a vertical cantilever with the wind acting horizontally to produce a moment at the base. As an example of the application of the method here shown I shall describe my method of designing a reinforced concrete chimney which was built to straddle a railroad track and in which the openings were large enough to pass a standard freight car. A solution of the stress due to moment and axial load in the shell on a section at the level of the openings follows. No attempt will be made here to give the derivation, since an analysis for shells without openings will be found in standard texts, such as *Reinforced Concrete and Masonry Structures* by Hool and Kinne. The analysis is here extended to include shells with openings.

The pipe will be considered to be a thin circular shell of radius R equal to the average radius of the shell. It is assumed that the concrete will take no tensile stresses, and that the vertical steel reinforcement will take no compressive stress.

The following notations will apply:

W = dead weight of pipe at section considered, in pounds

M = wind moment at section in foot-pounds = WE

E = eccentricity in feet = $\frac{M}{W}$

R = average radius of shell in feet

t = thickness of shell in inches

θ = central angle indicating position of neutral axis in degrees

α = central angle deduced from size of opening in shell, on the windward side

β = central angle deduced from size of opening in shell, on the leeward side

p = steel ratio

$n = \frac{E_s}{E_c}$ = ratio of modulus of elasticity of steel to that of concrete

F_c = unit compressive stress in the concrete at the radius R, in pounds per square inch

F_{cm} = maximum unit compressive stress in the concrete at the outer edge of the shell, in pounds per square inch

F_s = unit tension in the vertical reinforcement, in pounds per square inch

For convenience let

$$C = \frac{\sin\theta - \theta\cos\theta}{1 - \cos\theta}; \qquad D = \frac{\sin\beta - \beta\cos\theta}{1 - \cos\theta};$$

$$F = \frac{\sin\alpha + \alpha\cos\theta}{1 - \cos\theta}; \qquad G = \frac{\theta - \frac{1}{2}\sin 2\theta}{1 - \cos\theta};$$

$$H = \frac{2\left(\frac{1}{2}\beta + \frac{1}{4}\sin 2\beta - \sin\beta\cos\theta\right)}{1 - \cos\theta}$$

$$K = \frac{\pi - \theta + \frac{1}{2}\sin 2\theta}{1 - \cos\theta}$$

$$L = \frac{2\left(\frac{1}{2}\alpha + \frac{1}{4}\sin 2\alpha + \sin\alpha\cos\theta\right)}{1 - \cos\theta}$$

$$M = \frac{\sin\theta + (\pi - \theta)\cos\theta}{1 - \cos\theta}$$

Then it can be proved that

$$W = 24\,R\,t\,F_c\,[C - D - pn(M - F)] \quad . \quad [1]$$

$$WE = 12\,R^2\,t\,F_c\,[G - H + pn(K - L)] \quad . \quad [2]$$

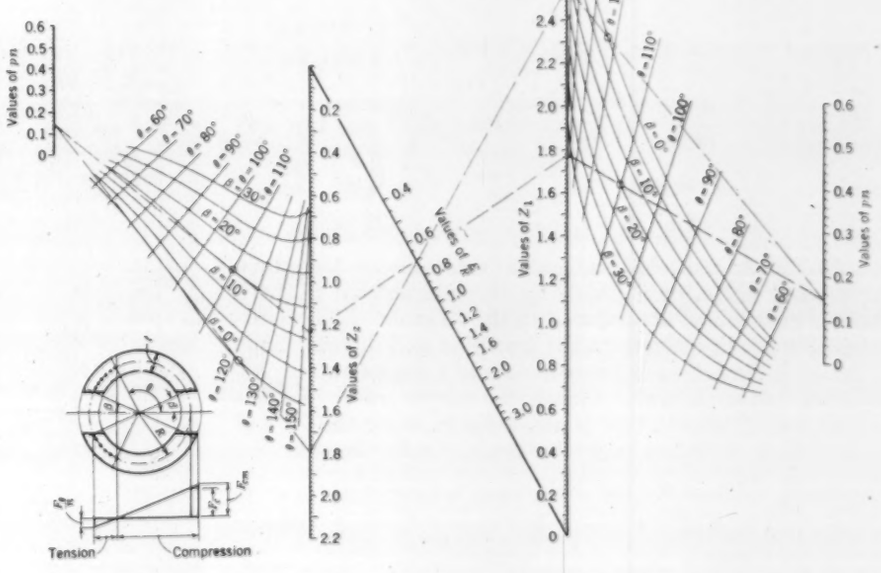

FIG. 1. DESIGN CHART FOR CONCRETE CHIMNEYS WITH TWO EQUAL OPENINGS

Equating Equations 1 and 2,

$$\frac{E}{R} = \frac{G - H + pn(K - L)}{2[C - D - pn(M - F)]} = \frac{Z_2}{Z_1} \quad [3]$$

and from Equations 1, 2, and 3, are secured

$$F_c = \frac{W}{12Rt} \times \frac{1}{Z_1} = \frac{W}{12Rt} \times \frac{E}{R} \times \frac{1}{Z_2} \quad [4]$$

$$F_{cm} = F_c \times \frac{\cos\beta - \cos\theta + \frac{t \sec\beta}{24R}}{1 - \cos\theta} \quad [5]$$

$$F_s = nF_c \times \frac{\cos\alpha + \cos\theta}{1 - \cos\theta} \quad [6]$$

FOR BOTH SINGLE AND DOUBLE OPENINGS

These six are general equations and are suitable for two openings in the shell opposite each other. It should be noted that the cosine in the second quadrant is negative. For a shell with a single opening the angle α equals zero; therefore F and L in Equations 1, 2, and 3 become zero. When it is desirable to determine the stress at a section where no openings exist, the angle β is taken equal to zero and D, F, H, and L in Equations 1, 2, and 3 are equated to zero.

In order to facilitate the solution of a given problem, charts, such as Fig. 1, may be used. This has been constructed for chimneys with two openings. For single openings a similar chart can be made.

With given values of $\frac{E}{R}$, pn, and angle β, the angle θ, which indicates the position of the neutral axis, can be found from the chart. The value of Z_1 can be found and used to calculate the stress in the concrete, F_c. By plotting the F_c value through the neutral axis, the numerical values of F_s and F_{cm} can be determined graphically or calculated as shown in the problems that follow.

FIRST EXAMPLE

In a given chimney, 300 ft high above the level of the bottom of the flue openings, the dead weight of the shell and lining is 3,000,000 lb and the moment due to wind loads is 27,300,000 ft-lb. R = 13 ft; t = 24 in.; p = 0.01; n = 15; and two equal openings making angle β = angle α = 15 deg. Then $E = \frac{27{,}300{,}000}{3{,}000{,}000} =$ 9.1 ft; $\frac{E}{R} = \frac{9.1}{13.0} = 0.7$; $pn = 0.01 \times 15 = 0.15$; and $\frac{W}{12Rt} = \frac{3{,}000{,}000}{12 \times 13.0 \times 24} = 800.$

In Fig. 1, the left-hand index line connects the given pn value and a point made by the intersection of the β and θ curves and is produced to the Z_2 scale. The right-hand index line connects the pn value and a point made by the intersection of the same β and θ angles and is produced to the Z_1 scale. When the proper value of θ has been found after a few trials, a line joining the Z_2 and Z_1 values will intersect the center index line at the given $\frac{E}{R}$ value.

In this problem, θ = 110 deg, and Z_1 = 1.78. Therefore $F_c = \frac{800}{1.78} = 449$ lb per sq in.

$$F_{cm} = F_c \times \frac{0.9659 - (-0.3420) + \frac{24 \times 1.035}{24 \times 13}}{1 - (-0.3420)}$$

= 464 lb per sq in.

and

$$F_s = 15F_c \times \frac{0.9659 + (-0.3420)}{1 - (-0.3420)}$$

= 3,140 lb per sq in.

SECOND EXAMPLE

As the second example, consider a shell with no openings, for which the given values are: $\frac{E}{R} = 0.7$; pn = 0.15; t = 17 in.; $\frac{W}{12Rt}$ = 1,100; and R = 13.0 ft. In this case, use the curve for angle β equal to zero degrees in Fig. 1. It is found that angle θ = 122 deg and Z_1 = 2.55.

Then $F_c = \frac{1{,}100}{2.55} = 432$ lb per sq in.

$$F_{cm} = 432 \times \frac{1 - (-0.5299) + \frac{17 \times 1}{24 \times 13}}{1 - (-0.5299)} = 446 \text{ lb per sq in.}$$

and

$$F_s = 15 \times 432 \times \frac{1 + (-0.5299)}{1 - (-0.5299)} = 1{,}990 \text{ lb per sq in.}$$

Evaluating Steel Storage Tanks

By JAMES G. O'BRIEN
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IN the valuation of steel storage tanks of various capacities for assessment purposes, the curves shown in Figs. 1, 2, and 3 were derived and found to be exceedingly helpful in obtaining quick approximate results.

The curve in Fig. 1 was derived from actual contract prices furnished by various steel and steel tank manufacturers and data from other sources. The index was obtained by a comparison between current contract prices and those of other years for tanks of similar capacity, and an

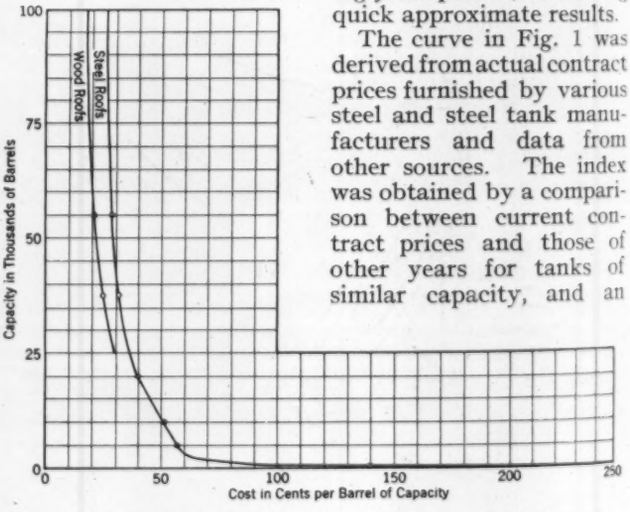


FIG. 1. BUYER'S COST OF STEEL TANKS ERECTED, PER BARREL OF CAPACITY

Based on an Index of 141.4 as of April 7, 1933, in which 1913 = 100, 1921 = 202, and 1925 = 206

average ratio taken of the index of these other years. This curve is most correct for tanks with a capacity of less than 55,000 bbl. It is suggested that the curves in Fig. 2 be used for larger steel roof-tanks. These curves were derived from approximate prices per pound for steel tanks erected. The data came from various steel and steel tank manufacturers and from other sources. The curves are adaptable for use for tanks with a capacity of from 25,000 bbl upwards. However, it is preferable that they be used for steel roof-tanks with a capacity of over 55,000 bbl.

USE OF COST CURVES

To use Fig. 2 read the capacity of the tank in barrels on the left and project the reading horizontally to the curves marked "Cents per Pound" and "Pounds per Barrel." Project the intersections vertically downward and read the "Costs in Cents per Pound" and the "Pounds per Barrel Capacity." Multiply these two figures and the capacity of the tank in barrels together to obtain the manufacturer's charge to the buyer of the steel tank erected as of April 7, 1933.

The cost curves in Figs. 1 and 2 are primarily applicable to Philadelphia and its vicinity. They can be used, however, with reasonable accuracy for the northeastern section of the United States. For other parts of the country the results must be multiplied by the ratio of the steel construction index of the section of the country desired to the index of Philadelphia. The curves can be adapted to a given year by multiplying the results by the ratio of the index of the year desired to the index, 141.4, used for Fig. 1. The results obtained represent approximate contract costs to the purchaser, not field or shop construction costs to the manufacturer. They do not include any earth excavation, foundation work, pipe work, or dike work.

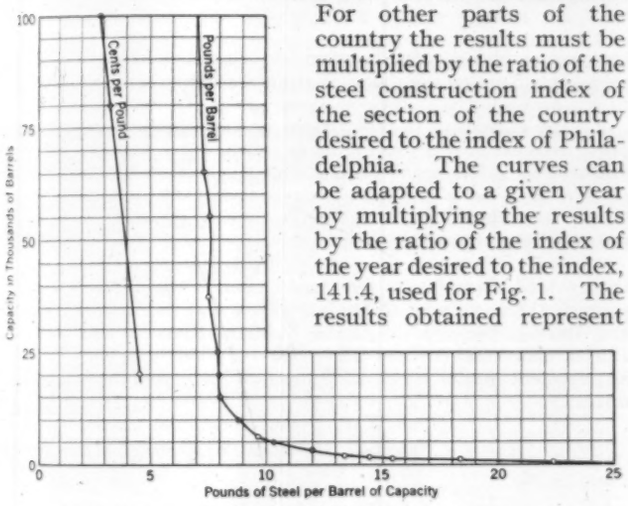


FIG. 2. BUYER'S COST OF STEEL TANKS ERECTED, AS OF APRIL 7, 1933

Based on the Product of Price per Pound of Steel Erected and Pounds of Steel per Barrel of Capacity

The curve in Fig. 3 was derived by plotting the depreciated percentages of the replacement value, which in the case of steel storage tanks would be the results obtained from the curves in Figs. 1 and 2. For the first year the percentage of depreciation is taken as the ratio of the total normal life, in years, to the sum of the series of numbers representing the years of normal life, that is, $\frac{25}{325} = 7.7$ per cent. The percentage of depreciation for the second year is equal to the ratio of one year less than the total normal life, in years, to the sum of the same series, that is, $\frac{24}{325} = 7.4$ per cent, and so on for the total normal life of the object. This method of computing depreciation gives a faster depreciation for the earlier years which, it is believed, is nearer to true conditions than the assumption of equal depreciation annually.

In Fig. 3 the depreciated value curve can be made approximately correct for a normal life other than 25 years by proportioning the years on the bottom line to the ratio of the adopted new normal life to that of 25 years. The results are then used as a new scale of years for the adopted new normal life. The use of the curve is adaptable to any object for which the "sum of series" theory is applicable.

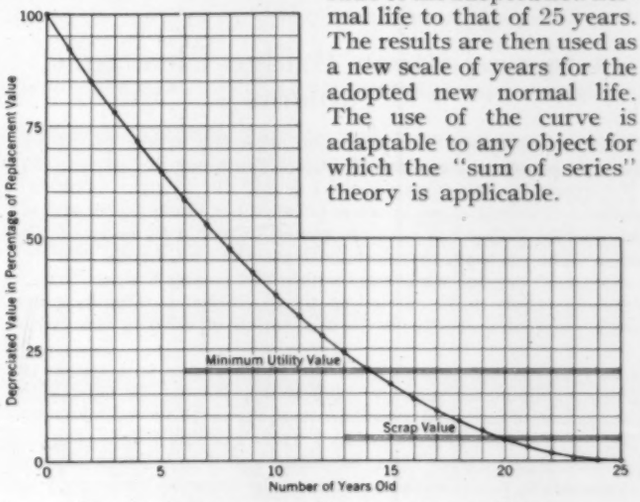


FIG. 3. CURVE FOR THE DEPRECIATION IN VALUE OF STEEL TANKS

Computed by the "Sum of Series" Method of Calculating Depreciation

Our Readers Say—

In Comment on Papers, Society Affairs, and Related Professional Interests

Salt Water in Seacoast and Island Wells

TO THE EDITOR: I should like to comment briefly on Mr. Riddel's article, "Excluding Salt Water from Island Wells," in the July issue of CIVIL ENGINEERING.

The theory of the relation between fresh and salt water on islands and sea coasts, which was applied by Badon Ghyben in Holland as long ago as 1887, but which is generally called the theory of Herzberg, is well known to ground-water geologists and engineers. Mr. Riddel's article is a valuable contribution to the literature on the subject, particularly because the water table is very close to sea level, with the result that the fresh-water body extends but a short distance—only 25 or 30 ft—below sea level. To judge from this depth, according to the theory of Herzberg, and assuming a density for the main body of salt water of 1.025, the water table should be somewhat less than one foot above sea level. It would be interesting to determine how this agrees with natural conditions. Because of the tides, careful observation with several automatic water-level recorders on wells and on the ocean would be required.

The author states that, "Provided no fresh water is taken from the area by pumping, the layer of fresh water should increase in depth every year." This, of course, is not a correct statement, and by inference in his following sentences he recognizes that fact. If a water-bearing formation is freely open to the ocean, the fresh water will accumulate, with increasing height above sea level and a proportionate depth below sea level, only to the point where a sufficient head has been built up to cause flow through the formation at a rate equal to the average annual rate of recharge from rainfall, as indicated by Mr. Riddel in his Fig. 3.

Aeolian sand has a higher frictional resistance than cavernous and honeycombed rock. This resistance requires a higher head to maintain flow and, at the same time, produces the fortunate effect of pushing down the salt water content some 25 or 30 ft below sea level. Under conditions similar to those described by Mr. Riddel, the fresh-water head should stand highest in large sand dunes, and accordingly the fresh water in these places should extend to the greatest depth below sea level.

To keep the record correct, it may be pointed out that one-third of an annual rainfall of 45 in. on an area of 8,770,000 sq ft will total only about 82 million gallons in a year instead of 96 million gallons a year (erroneously printed "mgd"), as stated by Mr. Riddel. This increases by just so much the difference between the assumed annual recharge on a basis of one-third of the rainfall and the reported annual pumpage of 100 million gallons. The difference is doubtless explained in part by the fact that the percentage of recovery of the rainfall reaching the water table—a very difficult factor to determine—is probably more than 33⅓ per cent; but it probably is also in part due to the fact that water is being drawn in from beyond the boundary of the area of 8,770,000 sq ft over which the wells are distributed.

To be correct, Mr. Riddel's diagrammatic Fig. 6 should show a cone of depression of the fresh water level around each pumping well, corresponding roughly to the inverted cones of salt water rising toward the well. The combined area of these cones of depression probably extends beyond the limits of the well field as defined. There must be a lowering of the water level in and near every well that is pumped in order to have flow of water toward the well. If the water-bearing material moves through open channels like those in cavernous limestone, the lowering may be so small that it will be detected only by careful measurement. This is especially true for small quantities of water, as in the installation described by Mr. Riddel.

The Nassau installation presents an excellent opportunity for observation of the problem of salt water, and it is to be hoped that Mr. Riddel will continue his observations and report them for the enlightenment of others engaged on similar problems.

DAVID G. THOMPSON
Senior Geologist, Division of Ground Water, U.S. Geological Survey
Washington, D.C.
August 31, 1933

Chicago River Bridge-Clearance Problems

TO THE EDITOR: In connection with the article by Messrs. Sultan and Ogden, entitled "Chicago Terminus of Lakes-to-Gulf Waterway," in the August issue, I should like to make a few comments. The inland water route between the Great Lakes and the Mississippi Valley has been completed to within 40 miles of Chicago. The problem, therefore, is to plan and ultimately provide a network of interior waterways within the greater Chicago area to serve the traffic to be carried on the main route. This local network must of necessity be an adaptation of existing canals and rivers.

It is important to note that the width of the rock cut in the Drainage Canal for some 15 miles is only 160 ft. In my opinion this section of the canal is too narrow to permit of successful navigation by river tows of standard lock size. For this purpose the rock cut should be at least 250 ft in width, and it is believed that enlargement to this dimension is out of the question for many years to come as the present and anticipated traffic is insufficient to support the great expense involved.

Accordingly, it will be necessary to break up river tows at the Brandon Road Pool. This condition suggests the use of specially designed towboats of smaller size for operations on the eastern end of the river. It thus would be possible to reduce the requirements for vertical bridge clearances to at least as low as 25 ft as far as river towboats are concerned. Open-top barges require not in excess of 12 ft of headroom, while covered barges for package freight would not require more than 25 ft. In the light of these facts, a vertical clearance of 25 ft for bridges over the Chicago Drainage Canal and other parts of the interior waterway in this district will meet all reasonable requirements for both towboats and barges.

Traffic on the main route will be of two kinds as far as type of carrier and terminal treatment are concerned. Bulk materials will be handled in barges of low headroom feeding local industries in full barge-load lots. Package freight will be handled in covered barges, largely in less than barge-load lots.

As regards source and destination, local traffic will be handled to and from industries located directly on the water front, or to industries back of the water front by means of trucking or switching. Through traffic will involve transfer between barge and truck, barge and railroad, or barge and lake vessel. It is believed that at least 75 per cent of the traffic will be of the bulk variety. Therefore bridge clearance will not be a factor in terminal service for this traffic. Of the package-freight movement, not over 20 per cent will be interchanged between lake vessels and barges. Since this is only 5 per cent of the total, the accessibility of masted vessels to the package-freight terminal locations is of minor importance.

For the movement of bulk freight on the inland waterway, the problem is to provide access to as much water front as can be arranged for. When this requirement has been met, the problem of providing transfer points for rail and truck and lake-vessel freight is a simple one, for the bulk-cargo barges may reach these points without bridge interference.

Channels with bridges having a vertical clearance of 25 ft should be provided through the Drainage Canal to the Chicago River, through the Sag Canal to Lake Calumet and the Calumet River and Harbor, through the Grand Calumet River to the Indiana Harbor Canal and Indiana Harbor, and ultimately to Gary. This arrangement would serve all important industrial areas in the district.

RUFUS W. PUTNAM, M. Am. Soc. C.E.
President, Maritime Engineering Corporation
Chicago, Ill.
August 19, 1933

New Economy in Motor-Vehicle Operation

TO THE EDITOR: The article by Mr. Paustian, in the July issue, describing the determination of tractive resistance by the use of a test car with gas-electric drive, is most timely. It is to be hoped that the author will give the results of the completed tests to the profession as soon as possible.

From the data presented in Mr. Paustian's Fig. 2, a number of tentative conclusions regarding the effect of road surface on gasoline consumption can be made. Accepting dry gravel and concrete as typical of intermediate and high-type surfaces, the gasoline consumption on intermediate surfaces is now shown to be about 10 per cent in excess of that on high-type surfaces, and not the 20 per cent shown in previous tests by T. R. Agg, M. Am. Soc. C.E., which have long been the bible of highway economists. This fact, together with the great change in the cost of tires that has taken place since the latter tests were made, violently upsets many of the estimates and some of the conclusions reached by previous contributors to CIVIL ENGINEERING.

It is no longer true that it costs 2 cents per mile more to operate a car on low-type, or earth roads than on high-type, or concrete surfaces, or 1 cent per mile more on gravel roads than on concrete.

At the 1932 Fall Meeting of the Society, Dean Agg presented a paper in which these differences were reduced to 0.94 cent and 0.80 cent, and it is my opinion that even these lowered values do not adequately reflect the present-day cost of tires.

Mr. Paustian's Fig. 2 shows further that, as regards gasoline consumption, bituminous treated gravel occupies a place about halfway between the high and intermediate types for speeds in excess of 35 miles per hr and very close to the high-type surfaces for lesser speeds. It is then certainly fair to competing types to compute the cost of gasoline and all dependent items on bituminous treated surfaces at 5 per cent in excess of corresponding costs on concrete. Bituminous treated gravel may be intermediate in first cost, but it should be put in a class by itself when vehicle operating costs are being considered.

Since the cost of vehicle operation on different surfaces is the fundamental factor in economic comparisons of such surfaces, almost all previous comparisons using Dean Agg's data are now out of date. In his address before the 1932 Fall Meeting of the Society, he stated, "that for economic studies it is on the conservative side to assume that the change from the lower type saves the automobile user about one-eighth of the operating cost of that type. This amounts to 0.8 per cent per mile of travel for the composite average automobile, on the basis of 1931 operating costs." This is considerably less than the $1^1/_4$ or 2 cents per mile employed by highway economists still using pre-depression estimates to prove the economy of expensive concrete pavements. Pavements are economical now in many places, but they are not as profitable or necessary under 1933 conditions of travel and costs as they were in 1928.

Further, if future estimated savings are to be capitalized now in order to justify present expenditures, the experience of the last few years suggests the necessity for a little caution in mortgaging the future. Bonds issued will have to be paid, but vehicle operating savings will fluctuate with the number of vehicles using the road, with the fuel economy of the motors in use, with the cost of gasoline, with the habits and general standards of living of the people, and with the development of new inventions. Today there are certain states that have such bonds, but they are experiencing difficulty in collecting the savings that some motorists evidently feel they cannot afford to make.

H. E. PHELPS, M. Am. Soc. C.E.
Professor of Highway Engineering
State College of Washington

Pullman, Wash.
August 31, 1933

Designing and Building Rock-Fill Dams

DEAR SIR: I was interested in Mr. Fraps' articles on the Arizona code for dams in the April and May issues, particularly the parts relating to rock-fill dams. In the design of this type of dam it should be kept in mind that it is essential that a circumferential joint, besides having vertical and horizontal joints, be laid around the perimeter of the mass structure about 10 ft from the rock foundations.

In 1914, I was consulting engineer on the construction of the Strawberry Dam, which was built by contract. The contractor used his own device for conveying the rock from the quarries to the dam. This consisted of five or six separate cableways, which impinged against a stopper in the air where the ropes joined over the center of the dam. The individual rocks coming down over the different cables tripped against this stopper and were automatically deposited in a cone on the body of the dam. The rocks were afterwards transferred by means of a derrick to their final positions in the structure.

Previously the rock for the construction of such dams was conveyed by cable, or hoists, and carefully deposited on different parts of the dam. This was a slow process. For later structures, such as the Bowman and Salt Springs dams, the rock was carried direct by truck from quarries near the main dam.

I have visited Strawberry Dam only once since its completion in 1916, and I am pleased that the settlement has been so slight. This structure is a clean rock-fill dam like the Salt Springs, Bowman, and Morena dams. The perniciousness of mingling dirt with rock on a rock-fill dam is well illustrated by the recent failure of the Castlewood Dam near Denver.

It is not practical to have a slope greater than 1.4 to 1 on the downstream face. I believe that a width of from 12 to 15 ft at the crest is adequate for a low rock-fill dam.

M. M. O'SHAUGHNESSY, M. Am. Soc. C.E.
Consulting Engineer
Public Utilities Commission

San Francisco, Calif.
September 1, 1933

Equations for Torsional Stresses in Error

TO THE EDITOR: Errors appear in my letter which was published in CIVIL ENGINEERING for September. In the lower half of the first column on page 533, the letter S, which appears a number of times in the denominator of fractions, should, in every case, be in the numerator. Also, the commas should be omitted from the equations for Y_f and Y_a; that is, they should read:

$$Y_f = \frac{9}{18} V_f = \frac{9}{18}\left(+ \frac{18}{33} S\right) = + \frac{9}{33} S$$

$$Y_a = \frac{18}{18} V_a = - \frac{18}{33} S$$

H. L. BOWMAN, M. Am. Soc. C.E.
Professor of Civil Engineering
Drexel Institute

Philadelphia, Pa.
September 8, 1933

Power Planning as the Basis of National Planning

TO THE EDITOR: In his very interesting and timely article in the September issue, Mr. Crane emphasizes the importance of "Planning for Planning." We are in the midst of a great historical upheaval. Only a few months ago we were upholding a doctrine of rugged and despotic individualism. Now we have cheerfully abandoned this laissez-faire policy. Under the leadership of the President himself, our country, which only recently served as a classic example of individualism, is economically and psychologically making great strides toward a new order, the foundation of which is laid on so-called planned economy. The end is probably at hand for free play of individual interests and natural economic forces.

Planning, according to Mr. Crane, ". . . is the exercise of collective intelligence, expressed through government, to guide human activity toward the common well-being." This definition, in my opinion, overlooks a serious objective of planning, that is, the direction in which human activity should be guided. To it, therefore, should be added, ". . . with maximum conservation of natural resources and minimum use of energy in general, and human energy in particular."

Human energy and the amount of developed power in man's possession are the essentials of modern progress. The more natural power serves a nation, the greater is its production, the higher its standard of living, and the less the necessity, in theory at least, of work for the maintenance and improvement of this standard. Development of power resources is the alpha and omega of modern economy, radiating from centers of energy.

It is easy to understand why Mr. Crane refers to Russia as an example of comprehensive planning where there exists the same aim

of ordered economy, the same technique of planning, with a great difference, of course, in political methods and social purpose, which are communistic. The first important step in the field of national planning was the preparation, in 1920, of the Master Plan, the so-called electrification of Russia. This plan included the building of thirty regional electric plants—of which 20 are run by fuel and 10 by water power—of a general capacity of 1,750,000 kw. These are centers of electrical energy for the industrial needs of the whole country.

In the United States the Federal Power Commission is now beginning to create a national plan for the development of water power. It seems that such a plan may be formulated only after consideration of other power resources and study of the present and future consumption of power. This leads in turn to the study of industries, agriculture, transportation, and in general the whole economic life of the country. If the problem is regarded as broadly as now seems possible, the United States will have its first, although perhaps not a very detailed, general plan of ordered national economy. This does not invalidate the correctness of Mr. Crane's theory of the necessity of a special commission for coordination of national planning. Whether this will grow around the planning work of the Federal Power Commission or whether it will be a separate organization, is a question of time and convenience.

In the Soviet Union a special State Planning Commission did not begin to function until five years following the preparation of the first national plan. This commission, which serves in an advisory capacity, includes 22 members and more than a thousand employees, most of whom are highly qualified specialists. There are a dozen departments in the commission, including a power department, which plays the most important rôle.

VALENTINE V. TCHIKOFF, Assoc. M. Am. Soc. C.E.

New York, N.Y.
September 4, 1933

Comments on Failures in Sheet Piling

TO THE EDITOR: In a letter in the August issue, Mr. Converse discusses the article by Professor Abbett and me in the May issue. In this discussion he makes several statements concerning sheet piling on which I should like to comment.

I agree with the two following statements: (1) that the failure of the anchored bulkhead in Industry Haven, Stettin, Germany, was in no way connected with its action as a free standing wall, and (2) that the Blum-Lohmeyer method of sheet-piling design is excellent. The first statement may be found on page 265 of our article, and the theory mentioned in the second statement is taught in a graduate course in soil mechanics that I offer at Yale University. I disagree, however, with the rest of the letter.

In discussing the Stettin problem, Mr. Converse states that, after a partial failure of the anchor, the wall bulged because a wall requiring a support at the top cannot stand without large deflections when this support is removed. In my opinion, the bulging of the wall and the gradual deflection of the tie-rods developed simultaneously, and at no time could the support be considered as "removed." Actually, the system—wall + tie-rods + anchor piles—acted as a frame placed in a dense compressible medium (earth) or filled with that material. The wall, which represented one of the vertical members of that ideal frame, had its lower end fixed. Therefore, when the overloaded tie-rods started to deflect, the top of the wall moved inside, and the central part of the wall span bulged outside.

It is hard to say what was happening to the interior of the soil mass when it was cut by the deflecting tie-rods, but this is immaterial since it is clear that the overstrained soil had to follow the deformed wall. Thus the bulging of the wall favored the deflection of the tie-rods, which otherwise would stop. In their turn the tie-rods developed a vertical downward reaction component, which produced a moment favorable to further buckling. I believe that if the wall had been more rigid the tie rods would not have been destroyed. Furthermore, the support given to the wall in Stettin by its tie-rods was always efficient—to a certain extent, admittedly. This follows from the fact that all the tie-rods, both safe and damaged, were held tight by the overloading earth mass. If the support had been removed, the potential energy accumulated at the top of the wall during the bulging would become free. The top of the wall would then incline, or even rush, toward the sea, an event which did not occur.

Unlike the wall in Stettin, which was not free, that represented in Fig. 5 of our article behaved in our opinion as a free structure. There is no settlement between crack *a* and the bulkhead, and crack *b* is higher than the adjacent ground surface, as it should be. In order to prevent the motion of a body, it is necessary to fasten it to something stable and not to another body which is also bound to move. This is our first conclusion: that a wall should be fastened to the stable section of the soil mass.

Reverting to the Blum-Lohmeyer method, I believe that this combined name, which I used in the beginning of this letter, is adequate for the following reason. In *Die Bautechnik*, 1931, No. 54, page 784, Dr. Lohmeyer published a review of a work by Dr. Blum entitled *Einspannungsverhältnisse bei Bohlwerken*. In that review Dr. Lohmeyer stated that the ideas of Dr. Blum were applied in both the references given by Mr. Converse. The method in question is very well discussed in English in the catalogue on sheet piling of the Carnegie Steel Company, Insert B, January 1933. Another comprehensive discussion in English of the Blum-Lohmeyer method is included in an article on sheet piling by S. Packshaw, published in *Civil Engineering* (London), also in January 1933.

DIMITRI P. KRYNINE, M. Am. Soc. C.E.
Research Associate in Soil Mechanics
Yale University

New Haven, Conn.
August 25, 1933

Data on Transportation Facilities Corrected

DEAR SIR: In Colonel Jonah's article in the August number, I note a statement to the effect that the De Witt Clinton locomotive of 1831 is on exhibition at the Century of Progress Exposition. The fact is that the locomotive exhibited is a reproduction of the original and was built to show at the Chicago fair of 1893.

It would be in the interest of accuracy if, in giving rates of speed, the distances for the records claimed were also given by the author. Records are referred to in the popular miles an hour unit, when they actually refer to speeds made for a period of seconds or minutes on a short tangent, and hence are not fairly comparable with the other quoted rates of actual miles run in an hour. Long before 1893, trains often ran 60 miles an hour for a few miles, but the actual scheduled time of the Empire State Express in that year was 51 miles an hour for 440 miles, including four stops. In 1876 a special train running from New York to San Francisco traveled 3,317 miles in 83 hr and 27 min, or about 40 miles per hour. Neither speed for short runs nor for long distances has improved much during a half century of progress.

It might also be noted that, while there may have been no hint of the automobile at the Chicago fair in 1893, there was quite a decided hint of it in Europe, and only two years later road races for motor cars with gasoline engines were inaugurated in France. It is therefore scarcely correct to infer that the automobile dates from 1904 and first appeared at the St. Louis fair. The author, as is proper, claims credit for our country for many contributions to the development of transportation, but the introduction of the automobile should not be included among them, although its development to revolutionize land transportation may well be credited to us.

HENRY WELLES DURHAM, *M. Am. Soc. C.E.*

Sandwich, Mass.
August 29, 1933

SOCIETY AFFAIRS

Official and Semi-Official

Charter for New Joint Agency Ratified

Efforts for Professional Development

AT ITS MEETING in Chicago, on June 27, 1933, the Board of Direction ratified the Charter and Rules of Procedure of the newly formed Engineers' Council for Professional Development. This new joint agency has been formed as a conference of enginering bodies for the purpose of enhancing the professional status of the engineer through the cooperative support of the national organizations directly representing the professional, technical, educational, and legislative phases of an engineer's life.

This organization plans to accomplish its purpose by promoting and coordinating efforts for higher professional standards of education and practice, greater solidarity of the profession, and greater effectiveness in dealing with technical, social, and economic problems. An immediate objective is the development of a system whereby the progress of the young engineer toward professional standing can be publicly recognized through the establishment of such qualifications as will enable him to meet minimum professional standards.

PROGRAM OF PROCEDURE

As stated in the Charter and Rules of Procedure, the program of the organization has four main phases, as follows:

1. Working out of further means whereby young men may be made to understand the responsibilities and opportunities of the profession, so that only those will seek entrance to it who have the requisite character and ability.
2. Formulation of criteria for colleges of engineering which will ensure to their graduates a sound educational basis for the practice of engineering.
3. Working out of plans for the further personal and professional development of young engineering graduates, and also of those without formal scholastic training.
4. Development of methods whereby engineers who have already met suitable standards may receive corresponding professional recognition.

COMMITTEES ORGANIZED FOR JOINT ACTIVITY OF SEVEN ORGANIZATIONS

Four standing committees are authorized, their names corresponding with the four main objectives, as follows: Committee on Student Selection and Guidance, Committee on Engineering Schools, Committee on Professional Training, and Committee on Professional Recognition.

Beside the Society, the participating bodies are the American Institute of Mining and Metallurgical Engineers, the American Society of Mechanical Engineers, the American Institute of Electrical Engineers, the American Institute of Chemical Engineers, the Society for the Promotion of Engineering Education, and the National Council of State Boards of Engineering Examiners. Six of these bodies have now ratified the Charter and Rules of Procedure; the seventh is expected to act in October. The first annual meeting of the new body will be held during the early part of October, when permanent officers will be elected.

In view of the great professional possibilities of this activity, every member will have a distinct interest in its development. Specific details will be available as soon as the formal preliminaries are fully completed.

HORATIO ALLEN
Fifth President of the Society, 1872–1873

Horatio Allen Scholarship Awarded

EACH YEAR Columbia University places at the disposal of the Society a scholarship in civil engineering. It has been named for the distinguished engineer, Horatio Allen, of the Class of 1823 at Columbia, who became (1872–1873) the fifth president of the Society. The scholarship, which is much coveted, includes tuition fees, a room in Furnald Hall, and a generous allowance for books and extra expenses. Its total value is about $700.

From the many applications received this year, the Society's Columbia Scholarship Committee recommended that the honor be awarded to Joseph T. Gollup, a 1933 graduate in civil engineering of the College of the City of New York. Its recommendation received the approval of the university authorities, and Mr. Gollup registered during September. He is to be congratulated on winning this honor.

Harrison P. Eddy, Official Nominee for President, 1934

CHOICE AS candidate for President of the Society for the forthcoming year, 1934, was made by the Nominating Committee, meeting in Chicago on September 25, in the person of Harrison P. Eddy, M. Am. Soc. C.E. Mr. Eddy thus becomes the "Official Nominee" to appear with other candidates for the remaining offices on the ballots to be canvassed in January.

By character, by engineering attainment, by interest in and service to the Society over many years, he is eminently fitted for this highest office in the organization. A more complete notice of Mr. Eddy's numberless activities and high professional qualifications will appear in a later issue.

An Unusual Issue of Transactions

A NUMBER of unusual experiences are in store for members who will receive PROCEEDINGS in the month of October. Those whose volumes of TRANSACTIONS come to them in paper covers will secure two volumes of TRANSACTIONS under the designation, "PROCEEDINGS, Part 2 and Part 3." This, with the regular Part 1 of PROCEEDINGS, will make a total of 2,383 pages received in one carton during the month.

Members who order their TRANSACTIONS either in cloth or morocco binding will receive Part 1 of PROCEEDINGS in an envelope as usual. Their regular TRANSACTIONS, Vols. 97 and 98, will then come to them in two parts in a single carton. Volume 97 will contain the report on the George Washington Bridge. Volume 98 will contain papers that have been discussed and closed in PROCEEDINGS during the past month, to a total of 31 papers. In addition there will be more than 200 pages of Memoirs of deceased members and the customary indexes by author and subject.

It is to be noted in passing that the prizes for papers published in TRANSACTIONS, to be awarded at the Annual Meeting next January, were based on papers in Vol. 96 and not the present Vol. 98. The Prize Committee for Vol. 98 has not yet been appointed, so that prizes for the papers contained in it cannot be awarded until January 1935.

REPORT ON GEORGE WASHINGTON BRIDGE

Of particular interest to members is Vol. 97. Not since December 1911, when Vol. 74 was mailed to the membership, has TRANS-

ACTIONS appeared on other than a very thin and transparent grade of Bible paper. In order to present the numerous photographs of the George Washington Bridge to the best advantage, Vol. 97 is published on a good grade of English finish paper.

One might ask how, in a year of unusual financial stringency, it has been possible to issue so much extra material. On completion of unusually large engineering projects, such as that of the George Washington Bridge across the Hudson River at New York, it is often customary for the engineering personnel to publish a memorial report of the project for a comparatively limited distribution. The cost of printing such a memorial is often considerable, since the work is done as a special job and in a relatively small edition. By electing to appropriate about the same amount of money for the use of the Society, the Port of New York Authority was enabled to secure separate reprints of this volume of TRANSACTIONS for its private distribution. With the expenditure of relatively little more, it has been possible for the Society to furnish all its members with a technical record of this milestone of engineering progress.

Thus the fine spirit of cooperation shown by the Port of New York Authority has made this valuable record available to every member of the Society without placing an additional burden on the Society's publication budget.

SCOPE OF PAPERS IN VOL. 98

Classified broadly, the papers in Vol. 98 of TRANSACTIONS deal with such subjects as arch bridges, highways, dams, soil research, waterways, reclamation, wire-rope design, truss bridges, subways, wind bracing, aerial surveying, steel towers, concrete, suspension bridges, locks, contracts, power analysis, and steel columns. This imposing list should be an answer to civil engineers who are prone to deplore an imagined tendency toward a separation of the profession into an infinite number of smaller parts.

A word should be said concerning parts of TRANSACTIONS that have not previously appeared elsewhere, such as the address by President Alonzo J. Hammond, delivered at the meeting of the Society held in Chicago on June 27, 1933. This noteworthy address deals with "A Century of Progress" in civil engineering, and will be well worth the time spent in reading it.

MEMOIRS

Those who have lost colleagues or acquaintances in the profession will wish to examine the section on Memoirs. The Society is continually pushing forward its program to make the list of professional Memoirs as complete as possible. It should be considered the duty of each member interested in the welfare of the Society to make every effort to supply missing Memoirs or information from which such Memoirs can be prepared.

As usual, the loss to the Society of especially valued and active members is witnessed in the author index, where in several cases it will be seen that a man has contributed to the technical knowledge of his profession right up to the moment of his passing, his Memoir appearing in the same volume. Memoirs of engineers of outstanding prominence in their profession are included, such as Col. W. A. Starrett, F. H. Newell, Gardner S. Williams, Coleman Sellers, Dr. Charles Warren Hunt, Dr. John R. Freeman, Prof. George Fillmore Swain, Gen. William Barclay Parsons, F. W. Skinner, and John A. Roebling. A limited number of separate reprints of each of these Memoirs is available at Society headquarters and will be mailed free of charge to any who request them.

STEEL COLUMN RESEARCH

After many years of intensive research, the Society's Special Committee on Steel Column Research has submitted its final report, which is published for the first time in Vol. 98 of TRANSACTIONS. This report concludes the work of the committee, organized in 1923, and is the third report on the subject. Its chairman, F. E. Turneaure, M. Am. Soc. C.E., has served in that capacity continuously since its inception. In the report are given results of tests on 71 columns, on lacing bars, and on the compressive strength of outstanding plates and flanges. All the columns had pivoted ends and were tested under eccentric loads. The results were compared with theoretical values, using the yield point of the material as a criterion of strength. All forms tested, except the laced channel section, gave results substantially equal to, or greater than, the theoretical values. The laced channel section was, in general, somewhat deficient. The lacing bars tested included flats, angles, and channels.

A considerable part of the report consists of a discussion of all the tests made by the committee and by former experimenters, and a correlation of these with theoretical values based on the yield-point strength of the material and certain known or assumed eccentricities. It is shown that where the eccentricity and degree of restraint of the column are known, the ultimate strength of the column is satisfactorily represented by the theoretical formula, using the yield-point strength as a basis. As a consequence, it is possible to develop rational column formulas for various materials and for various conditions. For ordinary values of the slenderness ratio, a parabola is a very close approximation to the theoretical formula. On this basis rational formulas are suggested for ordinary structural steel.

That this is an unusual mailing is attested by the fact that not since 1905, when TRANSACTIONS, Vol. 14, was sent to the members in six parts, has technical material been made available to members of the Society in such quantity.

Students Awarded Prizes

IN ACCORDANCE with their usual custom at commencement time, the various Local Sections have presented awards to engineering students graduating with high scholastic honors from neighboring colleges and universities. With the approval of the Board of Direction, the recipient of such an award is permitted to join the Society as a Junior, and the Local Section pays his initiation fee and dues for the first year. To date the following prize winners for 1933 have been reported to Society Headquarters:

NAME OF STUDENT	COLLEGE	LOCAL SECTION GIVING AWARD
Leigh Owen Gardner	University of Arizona	Arizona
William Murdock Avery, Ivan Lee Wissmiller	University of Illinois	Central Illinois
Robert Ziegler	University of Cincinnati	Cincinnati
Smith Adron Ketchum	University of Colorado	Colorado
Earl Gilman Beard	Armour Institute of Technology	Illinois
Alfred Blickensderfer	Rose Polytechnic Institute	Illinois
Corwin King Clickner	Northwestern University	Illinois
Louis Frank Fahrberger, Jr.	Lewis Institute	Illinois
Milton Turner Kenney	Purdue University	Illinois
Lloyd Albert Pfaff	University of Illinois	Illinois
Ross Nyman Brudenell	State University of Iowa	Iowa
Marvin Otto Kruse	Iowa State College	Iowa
George Bingly Schoolhart	Cornell University	Ithaca
Eugene Joseph Peltier	Kansas State College	Kansas State
Morris Bordner Uhrich	Lehigh University	Lehigh Valley
Leonard T. Lynch	Lafayette College	Lehigh Valley
Horatio Nash Ogden	Tulane University	Louisiana
William Alvin Blank	University of New Mexico	New Mexico
James Milton LeRoy	North Carolina State College	North Carolina
Robert Frederick Kreiss, Jack Lenhart	University of Minnesota	Northwestern
Norman Henry Hall, Wendell Lee Foster	University of Oklahoma	Oklahoma
Russel Francis Hurst, Jr.	Villanova College	Philadelphia
Louis V. Mahler	Drexel Institute	Philadelphia
Earl John Witmer	Drexel Institute	Philadelphia
Robert Henry Reisner	Oregon State Agricultural College	Portland (Ore.)
Leaton Lewis Hughes	Washington University	St. Louis
Cyril Thomas	Stanford University	San Francisco
Edgar Garbarini	University of California	San Francisco
Arthur Jerome Davidson	University of Idaho	Spokane
Roscoe Emerson Van Liew	University of Utah	Utah
Norman Phillips, Jr.	Virginia Polytechnic Institute	Virginia
Frank Elbert Calhoun	Washington and Lee University	Virginia
Walter Sumner Gard	University of Virginia	Virginia

Other awards of Junior membership in the Society were made at commencement time as follows: the Kruesi Prize to James Edward Teague, of the University of Tennessee; and the Frederick B. Robinson Prize to Herbert Edward Blakeman, of the College of the City of New York.

A Preview of Proceedings

Because of a crowded issue the publication of the paper on "Duration Curves," by H. Alden Foster, M. Am. Soc. C.E., previously announced to appear in "Proceedings" for September, has been postponed until the October issue. Other papers to be found in the October number deal with the analysis of unsymmetrical concrete arches; observation and study of the deformation of a reinforced concrete building during construction; tests to determine the horizontal pressure exerted by saturated fills; and a description of the combination sewerage and storm-water system for Columbus, Ohio.

Intercepting Sewers and Storm Stand-by Tanks at Columbus, Ohio

In TRANSACTIONS for June 1910, the Society published a paper by John H. Gregory, M. Am. Soc. C.E., entitled "The Improved Water and Sewage Works of Columbus, Ohio," which was presented by him at a meeting of the Society held on March 2, 1910. The present paper, "Intercepting Sewers and Storm Stand-by Tanks at Columbus, Ohio," to be published in the October PROCEEDINGS, is particularly interesting when studied in the light of the earlier paper. It is by Professor Gregory, with R. H. Simpson, Orris Bonney, and Robert A. Allton, Members Am. Soc. C.E., as co-authors. The two papers afford a valuable view of civic growth in sewerage work over a period of about 25 years.

In the forthcoming paper the program of sewerage and sewage disposal improvement recently constructed in Columbus, Ohio, at a cost of $3,621,000, is described. The intercepting sewers provide for considerably greater volumes of storm water than have usually been handled in such structures. In one of the sewers, for example, the capacity varies from 600 to 2,110 gal and the average is 1,900 gal per capita daily.

A significant and noteworthy part of the paper is that dealing with the storm stand-by tanks, which are the first of their kind to be placed under construction in the United States. The function of these tanks is to remove heavy solid and floating materials from excess storm-sewage flows before they are discharged into the rivers and streams. The intercepting sewers and the storm stand-by tanks described mark a decided advance not only in the art of sewerage and sewage disposal as practiced in the United States but also in the art of purifying and cleansing rivers. The works described in this paper are part of a comprehensive program to deal with the pollution of the Scioto River and Alum Creek. They consist of intercepting sewers having sufficient capacity so that overflows of storm sewage to the streams will be limited to a predetermined number per year. The paper also describes regulating chambers to control the quantity of storm sewage entering the sewers and storm stand-by tanks to give partial treatment to all storm sewage in the intercepting sewers over and above the quantity that can be handled by the sewage treatment works.

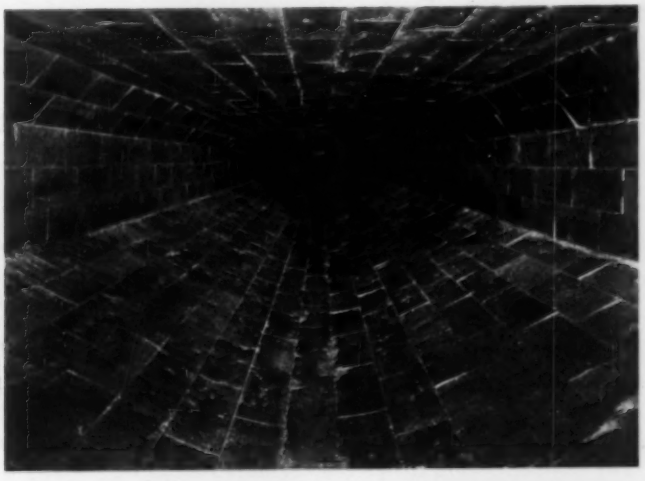
Some Soil Pressure Tests

ONE paper anticipated for this month's PROCEEDINGS, entitled "Some Soil Pressure Tests," by H. de B. Parsons, M. Am. Soc. C.E., is a sequel to a previous paper by the same author, "Hydrostatic Uplift in Pervious Soils," published in TRANSACTIONS in 1929. The research on which the forthcoming paper is based was made in order to verify the horizontal pressure due to a fill that had been completely saturated with water. Measurements were made of the horizontal pressures on a stationary bulkhead, exerted by bank sand and bank gravel. The fills were 7 ft deep, alternately dry, fully saturated with water, and drained. Results indicated that the horizontal pressures for a pervious soil were more dependent on the angle of internal friction than on that of repose.

Seven series of tests were made, and there were seven tests in each series, or 49 in all. In each series the soil was put into the testing apparatus without surcharge, its surface being always level and 7 ft above the bottom of the apparatus. It was then alternately irrigated and drained. The water was allowed to enter the back of the apparatus and rise on the bulkhead after seepage through the soil until it was flush with the surface. A summary of Professor Parson's findings, which appears at the end of the paper, will present many interesting topics for discussion.

Deformation of Steel Reinforcement During and After Construction

IT MAY BE expected that the paper by Sergius I. Sergev, instructor in civil engineering, University of Washington, Seattle, Wash., will call forth considerable discussion from engineers interested in the design of reinforced concrete structures. The arguments and conclusions advanced are the result of measurements made in the field on a full-scale building, Biology Hall, on the campus of the University of Washington. The author had about six primary objectives: (1) to observe the effect of



SEWERAGE WORKS AT COLUMBUS, OHIO

Above: Whittier Street Storm Stand-by Tanks; Left: Retaining Wall Along Scioto River, Over Olentangy-Scioto Intercepting Sewer; Right: Interior of Olentangy-Scioto Intercepting Sewer, 10 Ft 6 In. Wide and 5 Ft 6 In. High, Showing Vitrified Clay Liner Plates

shrinkage and flow of the concrete on the deformation of steel reinforcement; (2) to note whether the concrete and the steel act together during the early period of setting; (3) to note the actual stresses in the reinforcement; (4) to observe the magnitude of the initial stresses; (5) to compare the observed stresses with those computed and those allowable; and finally, (6) to note the variation in the deformations over long periods of time. For the purpose of this study, openings were left in the columns and in the beams at certain prescribed places. These permitted readings of deformation on an 8-in. Berry strain gage with long legs, so that readings could be taken to 1/40,000 part of an inch directly, and closer by interpolation. Basement-story columns, first- and second-story columns, and T-beams are treated under separate headings.

In conclusion, the author presents the claim that concrete and steel do not act together during the first seven days after pouring, and that initial stresses in the reinforcement are unusually high, varying with the richness of the concrete. He states further that framing a beam on one side of a column produces flexural stresses in that column. This is a field worthy of all the time spent on it. Discussion of Mr. Sergev's paper will be watched everywhere with interest.

ANALYSIS OF UNSYMMETRICAL CONCRETE ARCHES

AT A MEETING of the Society on February 4, 1925, Charles S. Whitney, M. Am. Soc. C.E., presented a paper, "The Design of Symmetrical Concrete Arches," which was subsequently published in PROCEEDINGS, discussed, and finally published in Vol. 88 of TRANSACTIONS in that same year. This was based on the work of Strassner, author of *Neuere Methoden*, published in Germany. Extending his studies since that time, Mr. Whitney now presents a companion paper entitled "Analysis of Unsymmetrical Concrete Arches." Those who study this paper in detail will wish to do so in connection with basic data published in the earlier paper.

The present paper deals particularly with a type of unsymmetrical concrete arch composed of halves of two symmetrical concrete arches. The thinner section of the arch is assumed to be at the highest point, and a different equation applies for the shape of the arch rib at each side of this point. This arrangement makes it possible to use curves and tables presented by the author in the earlier paper. It is stated by Mr. Whitney that this type of arch appears satisfactory even when the rib is very unsymmetrical and that, by simplifying the analysis, his method makes possible a more thorough study of the arch proportions. Inasmuch as it has been necessary to condense this paper considerably from its original length, he has consented to file the original manuscript complete for further reference in the Engineering Societies Library, at 29 West 39th Street, New York, N.Y.

News of Local Sections

COLORADO SECTION

At a meeting of the Colorado Section, held in Denver on June 17, officers for the ensuing year were elected as follows: E. L. Mosley, President; R. J. Tipton, Vice-President; and F. C. Carstarphen, Secretary-Treasurer.

GEORGIA SECTION

There were 20 present at a meeting of the Georgia Section held at the Atlanta Athletic Club in Atlanta on August 7. Discussion on the subject of a code for civil engineers occupied part of the session. The feature of the occasion was a talk given by H. F. Wiedeman, Sanitary and Hydraulic Engineer of Atlanta, on "The Water Supply of Atlanta and the Future."

PORTLAND (ORE.) SECTION

On August 24 the Portland (Ore.) Section and the Professional Engineers of Oregon gave a farewell dinner in honor of Harry S. Rogers, member of the Section and former dean of engineering at Oregon State College, who was preparing to leave Portland to take up his new duties as president of the Polytechnic Institute of Brooklyn. J. C. Stevens, consulting engineer of Portland, acted as toastmaster and introduced the various speakers, who told of their work with Dean Rogers and of the influence that he has had in engineering education. In response, Dean Rogers gave a brief résumé of his work at Oregon State College and stated some facts concerning the Polytechnic Institute of Brooklyn. There were 50 engineers present at the dinner.

SACRAMENTO SECTION

Varied and numerous activities have characterized the program of the Sacramento Section during the past summer. On Memorial Day 31 members and guests visited the North Star Gold Mine in Grass Valley. On June 6, Charles Derleth, Jr., Professor of Civil Engineering at the University of California, addressed a meeting of the Section on the subject, "The Golden Gate Suspension Bridge." And on July 18 the Section celebrated its five hundredth regular weekly meeting by holding a session attended by 11 of its former presidents, who gave informal talks.

SAN FRANCISCO SECTION

A dinner meeting was enjoyed by 94 members of the San Francisco Section on June 20. Entertainment in the form of recitations was provided during dinner. After a routine business session two speakers were heard. These were Perry Byerly, seismologist at the University of California, and Walter L. Huber, consulting engineer of San Francisco. Both spoke on subjects of interest to engineers in connection with the recent California earthquake and emphasized the need of earthquake-resisting structures.

Student Chapter News

AGRICULTURAL AND MECHANICAL COLLEGE OF TEXAS

During the past school year representatives of the Agricultural and Mechanical College of Texas Student Chapter attended the fall meeting of the Texas Section of the Society at Mineral Wells. The Chapter also held 17 meetings of its own, with an average attendance of 35 at each meeting. At some of these sessions motion pictures were shown through the courtesy of commercial firms, and other meetings were addressed by members of the Chapter and faculty and by outside speakers.

CALIFORNIA INSTITUTE OF TECHNOLOGY

In addition to holding nine meetings of their own, the members of the California Institute of Technology Student Chapter have, during the past year, attended meetings of the Los Angeles Section of the Society, whenever it was possible to do so. Of particular importance was the meeting held on May 10, when the Chapter was host to the Los Angeles Section and provided the entire program of entertainment. Several inspection trips have been made to nearby points of engineering interest, including Pine Canyon Dam, the Pasadena Sewage Farm, and Boulder Dam. In all, 414 members participated in the various events.

CASE SCHOOL OF APPLIED SCIENCE (STADIA CLUB)

Lectures by students, members of the faculty, and outside speakers formed the nucleus of the program of activities of the Case School of Applied Science Student Chapter during the past year. On January 24 the members were guests of the Cleveland Section of the Society at a meeting at which the principal speakers were Robert Hoffmann, consulting engineer for the Department of Public Works, City of Cleveland, and W. A. Stinchcomb, Director of the Cleveland Metropolitan Park Board. On one occasion an illustrated lecture loaned by the Society was enjoyed, and on another W. E. Wickenden, President of the Case School of Applied Science, addressed the members. There were nine meetings held, with a total of 250 in attendance.

COLLEGE OF THE CITY OF NEW YORK

There were 22 meetings held by the College of the City of New York Student Chapter during the school year just completed. These attracted a total attendance of 855. G. E. Barnes, Assistant Engineer of Design for the Department of Sanitation; P. L.

Gerhardt, Industrial Consultant for the Port of New York Authority; D. B. Steinman, consulting engineer of New York, N.Y.; and Robert Ridgway, Past-President of the Society and consulting engineer for the Board of Transportation, City of New York, were among the noted engineers who spoke. At the meeting held on December 22, Professor McLoughlin, the faculty sponsor, announced that he was establishing a prize of a Junior Membership in the Society. The award, designated as the Frederick B. Robinson Prize by the donor, is to be made annually to the student member who has done the most for the Chapter during the year.

COLUMBIA UNIVERSITY

During the past year the members of the Columbia University Student Chapter enjoyed several meetings and social gatherings. One interesting session was addressed by Charles P. Berkey, Professor of Geology at Columbia University, who chose for his subject, "How Geology Affects the Location of Dams in The United States."

IOWA STATE COLLEGE

An unusually successful year for the Iowa State College Student Chapter has come to a close. In all, 19 meetings were held, with a total attendance of 2,000. On several of these occasions illustrated lectures furnished by the Society were presented. Other meetings were addressed by various well known engineers. Among these were the following: C. C. Coykendall, Administration Engineer for the State Highway Commission; Robert Ridgway, Past-President of the Society and consulting engineer for the Board of Transportation, City of New York; Willard T. Chevalier, of the McGraw-Hill Publishing Company; and H. J. Gilkey, professor and head of the Department of Theoretical and Applied Mechanics, Iowa State College.

JOHNS HOPKINS UNIVERSITY

Several joint meetings with the Maryland Section of the Society and with other engineering societies of Baltimore have been held by the Johns Hopkins University Student Chapter during the past year. Among the speakers who addressed meetings of the Chapter were W. T. Ballard, Vice-President of the J. E. Greiner Company; F. P. McKibben, consulting enginer, of Black Gap, Pa.; and Dr. J. T. Singewald. In March a dance was held in conjunction with student chapters of other engineering societies at the university. In the same month a trip was made to the steel fabricating shop of the Scully Steel Products Company.

LEHIGH UNIVERSITY

This school year has been one of the most successful in the history of the Lehigh University Student Chapter. Three of the Society lantern lectures were presented, and numerous interesting speakers were heard. Among these were Robert Ridgway, consulting engineer for the Board of Transportation, City of New York; Thaddeus Merriman, Chief Engineer of the Board of Water Supply, City of New York; and Edward H. Rockwell, Professor of Civil Engineering at Lafayette College. One meeting of the Chapter was addressed by H. S. Crocker and George T. Seabury, respectively Past-President and Secretary of the Society, both of whom discussed various aspects of Society affairs. In November the members of the Chapter made a three-day inspection trip to New York City, where they visited the Wards Island Sewage Disposal Plant, the Holland Tunnel, the Port of New York Authority Inland Terminal No. 1, and other projects of engineering interest.

LOUISIANA STATE UNIVERSITY

Monthly, and sometimes semi-monthly, meetings have been the most important feature of the program of the Louisiana State University Student Chapter during the past year. G. P. Rice, consulting engineer of New Orleans, addressed one meeting. The other 18 sessions were devoted to lectures by student members, who chose various timely topics for discussion.

MANHATTAN COLLEGE

A number of illustrated Society lectures were presented at the 1932–1933 meetings of the Manhattan College Student Chapter, and a careful analysis and study was made of each. Several of the bi-weekly meetings were addressed by outside speakers—among them the following: Arthur G. Hayden, Designing Engineer of the Westchester County Park Commission; Raphael C. Smith, former president of the Bronx Chapter of State Professional Engineers; and John C. Riedel, an engineer in the Bureau of Sewers, Borough of Brooklyn.

MASSACHUSETTS INSTITUTE OF TECHNOLOGY

The annual report of the Massachusetts Institute of Technology Student Chapter indicates that the activities of the Chapter during the past year have been very successful. In all, nine meetings were held, with a total attendance of 662. At four of these sessions films showing progress on various current engineering projects were presented through the courtesy of the U.S. Department of the Interior. Among the well known engineers who addressed the other meetings are the following: James W. Rickey, Chief Hydraulic Engineer of the Aluminum Company of America; Asa H. Morrill, Chief Engineer of the Maine Central Railroad and Portland Terminal Company; and Harrison P. Eddy, consulting engineer of Boston.

MICHIGAN COLLEGE OF MINING AND TECHNOLOGY

The Michigan College of Mining and Technology Student Chapter held seven meetings during the year, 1932–1933, with a total attendance of 193. The programs of entertainment for these occasions were divided between the presentation of student papers, the showing of Society lantern slides, and lectures given by outside speakers. The latter included Mr. Vivian, Chief Engineer of the Calumet and Hecla Consolidated Copper Company, and Clifton Cole, who described the possibilities for American engineers in South America.

MISSOURI SCHOOL OF MINES AND METALLURGY

Meetings held by the Missouri School of Mines and Metallurgy Student Chapter during the past year have been very well attended. In addition to talks given by members of the Chapter and the faculty, several outside speakers have been heard. These include H. C. Beckman, District Engineer for the U.S. Geological Survey, and Captain Harloe, of the Corps of Engineers, U.S. Army.

UNIVERSITY OF MISSOURI

The University of Missouri Student Chapter has enjoyed a splendid year from the standpoint of both attendance at meetings and spirit of cooperation and interest shown by the members. Early in May a trip was made to Bagnell Dam and the Jefferson City water supply. At the eight meetings held during the year good use has been made of illustrated lectures, some of which were furnished by the Society and others by Stone and Webster.

UNIVERSITY OF NEVADA

On April 11 the University of Nevada Student Chapter enjoyed a trip to Boulder Dam, where the members inspected the project and heard a talk on construction given by Boyd Armiger, Construction Engineer on the U.S. Post Office in Reno. At the 1932–1933 monthly meetings of the Chapter interesting illustrated lectures have been heard on a variety of engineering subjects.

UNIVERSITY OF NEW MEXICO

At each of the meetings of the University of New Mexico Student Chapter held during the past year, one of the senior members of the Chapter presented a paper or gave a talk on some civil engineering subject of special interest. Besides the monthly meetings, the Chapter took several short inspection trips to places of engineering interest. These trips were made under the guidance of its faculty sponsor, J. H. Dorroh, Professor of Civil Engineering at the university.

UNIVERSITY OF NORTH DAKOTA

In addition to six private meetings held by the University of North Dakota Student Chapter during the past school year, there were five joint meetings sponsored by the other engineering societies at the university, which the members were invited to attend. Among those who spoke at these meetings were Boyd Begg, of the Raymond Concrete Pile Company, and Earl J. Fennell, of the U.S. Geological Survey. On May 19 an Engineers' Day attracted an enthusiastic gathering.

UNIVERSITY OF PITTSBURGH

Almost every branch of the profession of civil engineering was represented on the list of topics chosen for discussion by the University of Pittsburgh Student Chapter during the school year, 1932-1933. There were 27 meetings and a total attendance of 2,213. Lectures by student members have been eliminated by the Chapter, and guest speakers of merit were heard at most of the meetings. These included N. B. Jacobs, Vice-President of Morris Knowles, Inc., of Pittsburgh; Winters Haydock, Directing Engineer, Department of City Transit, Pittsburgh; and L. C. McCandliss, Professor of Civil Engineering at the University of Pittsburgh. The U.S. Bureau of Mines furnished interesting motion pictures that were shown upon four occasions. Besides these activities the Chapter sponsored numerous social events.

UNIVERSITY OF SOUTH CAROLINA

During the school year, 1932–1933, the University of South Carolina Student Chapter held eight regular and three special meetings. Other meetings were devoted to a seminar engineering class conducted under the auspices of the Chapter, and outside lecturers and illustrated lectures furnished by the Society were enjoyed at some of the seminar sessions. In all, three inspection trips within the state were made as follows: the Saluda hydro-electric plant, the Parr Shoals hydro-electric and steam power plant, and the city water works of Columbia.

UNIVERSITY OF TENNESSEE

During the past year the University of Tennessee Student Chapter held 15 meetings, with a total attendance of 193. These sessions were addressed by students, members of the faculty, and outside speakers, who covered a variety of interesting subjects, ranging from matters of purely local engineering importance to those affecting the profession as a whole. At some of the meetings the Society's lantern slide lectures were presented.

UNIVERSITY OF TEXAS

The University of Texas Student Chapter reports that it has just completed an unusually successful and pleasant year. In the fall semester field trips were made to limestone quarries and a brick manufacturing plant near Austin, and in the spring a two-day trip included visits to the lime and natural rock asphalt quarries at New Braunfels and Uvalde, and to the plant of the Republic Portland Cement Company in San Antonio. Among the engineers who addressed meetings of the Chapter are the following: P. A. Welty, Manager and Chief Engineer of the Bexar-Medina-Atascosa Counties Water Improvement District 1, and San Antonio Suburban Irrigated Farms; T. U. Taylor, dean of the School of Engineering of the University of Texas; and J. P. Exum, of the Texas State Highway Department. A dance held in the spring semester drew a gathering of 190.

UNIVERSITY OF VIRGINIA

Early in the past semester John L. Newcomb, dean of the Department of Engineering, of the University of Virginia, and acting president of the university, gave a talk before the University of Virginia Student Chapter, in which he touched on the history, purpose, and activities of the Society. At other meetings during the year a series of motion pictures on various civil engineering subjects was shown, and the entire student body invited to attend.

UNIVERSITY OF WISCONSIN

Several outside speakers addressed the University of Wisconsin Student Chapter during the school year recently completed. Among these were the following: H. D. Blake, of the Wisconsin State Highway Commission; E. E. Parker, City Engineer of Madison, Wis.; L. F. Van Hagan, Professor of Railway Engineering at the University of Wisconsin; and C. N. Ward, an engineer with Mead and Seastone, of Madison, Wis.

UNIVERSITY OF WYOMING

There were 96 in attendance at the eight meetings of the University of Wyoming Student Chapter held during the past year. Lectures supplied by the Society and presented by members of the Chapter or the faculty comprised the program for most of these sessions.

VANDERBILT UNIVERSITY (ROBERT H. MCNEILLY)

Besides 14 regular meetings, the Vanderbilt University Student Chapter held several informal "get togethers" during the school year, when they entertained all the engineering students enrolled at the university. Several inspection trips were also enjoyed. These included a visit to the Muscle Shoals Dam, where the power and nitrate plants, as well as the dam, were inspected, and a week-end surveying party spent with the Summer Surveying Company at Bon Air, Tenn. The showing of motion pictures featured several of the meetings held by the Chapter. Several of these films were obtained through the courtesy of Stone and Webster and the Caterpillar Tractor Company.

VILLANOVA COLLEGE

The Villanova College Student Chapter has concluded an interesting year of meetings, lectures, and trips. It has long been an ambition of the Chapter to have its sessions addressed by student members, and the past year saw the realization of this ambition. A large delegation from the Chapter attended the Fall Meeting of the Society at Atlantic City, N.J., on October 7, which was Student Chapter Day. Later an interesting trip was made to the Philadelphia Navy Yard, where 25 members of the Chapter made a complete tour of the shops, factories, and ships that were open for inspection.

VIRGINIA MILITARY INSTITUTE

The Virginia Military Institute Student Chapter reports a 100 per cent enrollment of those in the institute eligible for membership. There were 16 meetings held during the school year, with a total attendance of 1,200. These sessions afforded the members of the Chapter an excellent opportunity to gain proficiency in public speaking, as 55 of them were represented on the various programs. Some of these talks were given in conjunction with the lantern slides furnished by the Society.

VIRGINIA POLYTECHNIC INSTITUTE

The Virginia Polytechnic Institute Student Chapter has held monthly meetings during the past year. On such occasions the members have had the opportunity to hear various interesting speakers. These include Franklin P. Turner, Principal Assistant Engineer of the Norfolk and Western Railway Company, and R. B. H. Begg, Professor of Civil Engineering at Virginia Polytechnic Institute.

WASHINGTON AND LEE UNIVERSITY

Motion picture lectures on various timely engineering subjects were enjoyed by members of the Washington and Lee University Student Chapter during the past year. At one meeting E. G. Rice, Divisional Engineer of the Chesapeake and Ohio Railway Company, spoke interestingly on the subject of tunneling. Other sessions were addressed by members of the Chapter and the faculty.

WORCESTER POLYTECHNIC INSTITUTE

Several successful meetings were held by the Worcester Polytechnic Institute Student Chapter during the school year. At one of these J. A. Johnston, District Engineer for the Massachusetts State Highway Department, gave an interesting talk on his work, and on another occasion Joseph A. Tosi, president of the Worcester Section of the Society, spoke on the subject, "The Engineer and the Contractor." Another talk that was much enjoyed was one on engineering experiences in South America, given by an alumnus.

YALE UNIVERSITY

On April 11 the Yale University Student Chapter sponsored a public lecture by Robert Ridgway, Past-President of the Society and consulting engineer of the Board of Transportation, City of New York. Mr. Ridgway lectured to a capacity audience of more than 500 on the subject of the Boulder Dam. Earlier in the season the Connecticut Section of the Society entertained the senior class members of the Chapter at dinner.

ITEMS OF INTEREST

Engineering Events in Brief

Civil Engineering for November

IN ORDER to make the highway dollar go as far as possible, engineers are giving their attention to means by which adequate highway bridges can be constructed at low cost. In an article to appear in the November issue, Searcy B. Slack, M. Am. Soc. C.E., states that material savings in both first cost and maintenance of such bridges can be obtained by using simple designs; by employing continuous-beam, rigid-frame, and cantilever construction wherever possible; by using open end-bents in place of abutments; and by combining creosoted timber piles and caps with steel joists and concrete floors.

Many engineers will welcome the article by R. P. Pennoyer, Assoc. M. Am. Soc. C.E., in which he presents the elements of a design method for steel sheet-piling bulkheads. The subjects treated include earth pressures, economical depth to which sheet piles should be driven, calculation of anchor loads, and selection of pile cross section in order best to hold the load behind the bulkhead.

A vexing problem confronted the City of St. Louis in connection with a railroad grade crossing at the intersection of Union and Lindell boulevards. As described by L. R. Bowen, M. Am. Soc. C.E., a thoroughly satisfactory solution was found by depressing the tracks through Forest Park. The excavated material was formed into embankments on each side of the tracks, which were sodded and planted.

To prevent uplift under the gravity abutments of the Calderwood Dam on the Little Tennessee River, and to stop leakage, an impervious curtain was formed by forcing grout into the rock. The article by J. B. Hays, M. Am. Soc. C.E., explains the geologic conditions at the site, describes the methods used to place the grout, and comments on their relative effectiveness.

Other articles—on the design of reinforced concrete tanks, on the commercial operation of a large quarry, on the permeability of concrete to water at high pressure, and on a surprising type of construction used in a historic French chateau—are in preparation and are expected to appear in the November issue.

Combining Timber and Concrete in Bridge Construction

DURING the summer of 1932 the Oregon State Highway Department began a series of investigations looking toward the development of a new type of short-span bridge that would combine concrete and treated timber in such manner as to utilize each material to its maximum advantage. The type being used in Oregon introduces a shear connection between the timber stringers and the concrete deck and provides an overhanging roadway to protect the wooden substructure from the elements and from fire hazard.

Tests to destruction were made on 22 full-size composite beams. These tests included the repeated application of loads. Other tests were made to determine the effect of thermal changes. Still others concerned selection of timber for such bridges and the proper treatment of it to reduce checking and splitting. One of the chapters of the report on these tests compares the cost of bridges of this type with that of other types normally used for short-span highway structures.

A limited number of copies of this report will be available to interested members on application to C. B. McCullough, M. Am. Soc. C.E., Assistant State Highway Engineer, Salem, Ore. Also, a copy has been filed in the Engineering Societies Library, 29 West 39th Street, New York, N.Y., and may be used for reference by engineers in the vicinity.

Lantern Lectures for Student Chapters

AT THIS SEASON, when the colleges are opening and the Student Chapters affiliated with the Society are becoming active, it is appropriate to call attention to the series of lantern lectures available to all student groups. These have been used by many Chapters to supplement the more formal classroom instruction. Under the direction of the Committee on Student Chapters, this service was initiated in the year 1929. It has been received most cordially, and has proved of value in colleges all over the country.

Reservations for one of these lectures may be made at any time. The Society pays the postage both ways if the material is used by a Student Chapter. Each subject is illustrated by a number of lantern slides—from thirty to sixty in a set. These are accompanied by a mimeographed lecture describing each slide in order. The slides and lecture are sent well in advance of the date when they are to be used, so that the student who is to present the material may familiarize himself with the topic and expand the description by personal study.

Those students who are specializing in the design of bridges will be interested in the two lectures which describe this department of engineering construction. The bridges treated are the Carquinez Strait Bridge, of cantilever type, in California, and the suspension span which links the city of Florianopolis, in Brazil, to the mainland. Another lecture illustrates the design of the multiple-dome Coolidge Dam, in Arizona.

The Holland Vehicular Tunnel, linking New York City with New Jersey, is notable not only for peculiar difficulties of excavation, but for new developments in ventilating apparatus. This tunnel forms the subject of one of the lectures. The subject of another is the eight-mile Cascade Tunnel, on the Great Northern Railway, used to visualize the methods of cutting a railroad tunnel through solid rock.

Water supply is another field of engineering in which the civil engineering student is actively interested. The cities of New York and San Francisco have both had their special problems in gaining an adequate water supply. Each is described in a well illustrated lecture in this series. Power development and transmission, although the special province of the electrical engineer, involve the constructive energies of the civil engineer. Recent developments at Niagara Falls form the subject of another lecture; and the Wilson Dam and power plant at Muscle Shoals are also presented, as well as the Conowingo hydro-electric plant, which furnishes power for the city of Philadelphia. The lecture on the Hetch Hetchy water supply of San Francisco takes up also the power plant connected with the project.

Flood control is another important branch of engineering. Two regions where this has been successfully carried on are in the Miami Flood Control District of Ohio and in the basin of the Mississippi River. The various methods used and the results obtained in each case are described in two of the lectures.

Maintenance of parks and parkways is coming more and more to the fore as a serious interest of the civil engineer. The development of Westchester County, the suburban home of many New York commuters, has attracted attention all over the country. We are fortunate in being able to include a description of this parkway system among the lectures.

A letter is being sent to all secretaries of Student Chapters giving the titles of the lectures in the series, and suggesting that reservations be filed promptly, so that the particular material desired may be available when needed.

Better Wrapping for Foreign Mail

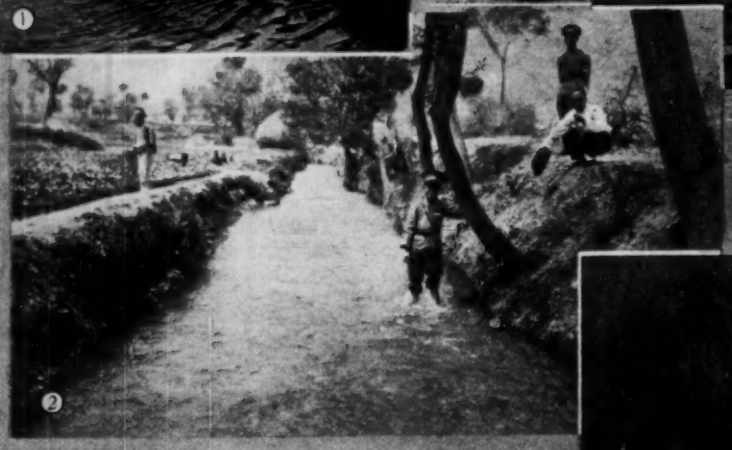
IN THE PAST, members abroad have not always received their copies of CIVIL ENGINEERING in the best of condition. Therefore a new method of mailing to foreign destinations has been adopted. For the usual flat manila envelope, formerly used for all copies and still used for those delivered in the United States, a full-length wrapper, in which the copy is tightly rolled, has been substituted. Although a slight increase in cost results, it is felt to be justified.

Modern Irrigatio

Sources of Supply

PHOTOGRAPHS BY O. J. TODD, M. AM. SOC. C

1. Kwan Hsuing Sau Spring, near Pingyong, Furnishes 100 Cu Ft per Se
2. A Fine Stream from Tung Tze Sau Springs
3. Diversion Weir to Fu Shan Canal
4. Diversion Dam on Fen Ho River, near Chaocheng
5. Wei Pi Irrigation Project
6. Yellow River Gorges North of Yumenkuan
7. Lower End of Aqueduct, West Main Branch, Wei Pei Project

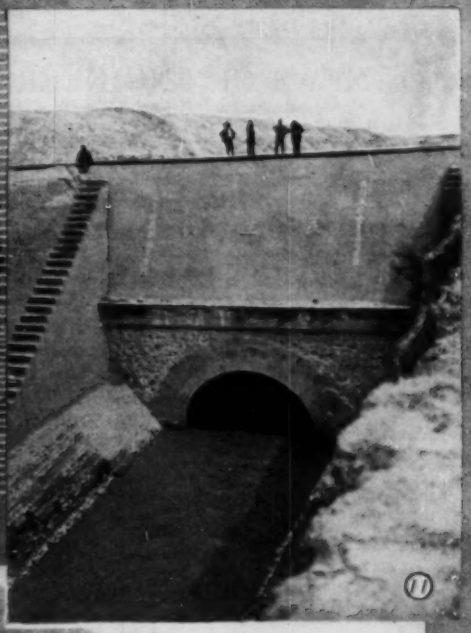


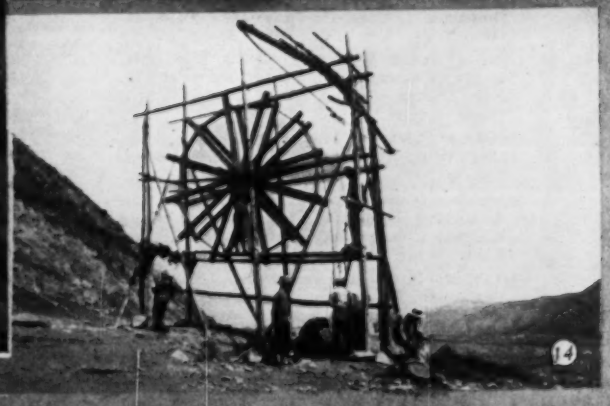
China Pictured

...neering Construction

...CHINA FAMINE RELIEF, U.S.A., Inc.

8. Spring at Shen Tao, Shansi
9. Concrete Aqueduct Over West Main Branch, Wei Pei Project
10. Road Building on Yuanshow Hill
11. Flood Water Structure Over Wei Pei Canal, Mushanyan
12. Yellow River at Yumenhsi
13. Drop on North Main Canal of Wei Pei Project
14. Drilling for Bedrock at Dam No. 2 on Pan Ho River





NEWS OF ENGINEERS

From Correspondence and Society Files

PAUL A. HARTUNG has received a U.S. Forest Service appointment as superintendent of Tehachapi Camp, at Keene, Calif.

RALPH A. TUDOR has been promoted to the position of Senior Designing Engineer on Bridges in the Division of Highways, State (of California) Department of Public Works. He is at present engaged on the plans for the San Francisco-Oakland Bay Bridge and has his headquarters in San Francisco.

JOHN S. DAVIS has accepted the position of Chief Field Engineer with the Southern Appalachian Development, of Anniston, Ala.

DON JOHNSTONE, formerly a surveyman for the U.S. War Department in the District Engineers' Office at Kansas City, Mo., has now become connected with the U.S. Waterways Experiment Station at Vicksburg, Miss.

T. R. JACOBI has resigned as Progress Engineer for Ulen and Company, at Eagle Pass, Tex., to enter the employ of the Indiana State Highway Commission, with headquarters in Fort Wayne, Ind.

G. ALBRO HALL has been made Superintendent Chemist of the new Norwalk (Ohio) Sewage Treatment Works. Formerly he was Sanitary Chemist for George B. Gascoigne, of Cleveland, Ohio.

W. SCOTT JOHNSON has resigned his position as Chief Public Health Engineer in the Missouri State Board of Health, with headquarters in Jefferson City, to become Sanitary Engineer, Department of Public Welfare, Division of Health, City of St. Louis, Mo.

JOHN R. ZEHNER has accepted a position as field superintendent in the construction and equipment department of Montgomery Ward and Company, of Chicago, Ill. He was previously with the Turner Construction Company, of Brooklyn, N.Y.

L. C. TSCHUDY has been made Supervising Engineer in the Bureau of Agriculture Engrs., with headquarters in La Crosse, Wis.

GEORGE E. BARNES, formerly Assistant Engineer of Design in the New York City Department of Sanitation, has been appointed Professor of Hydraulic and Sanitary Engineering and head of the Department of Civil Engineering at the Case School of Applied Science, Cleveland, Ohio.

R. C. BOOTH has recently been appointed Division Engineer in charge of work in Division No. 1 of the Metropolitan Water District of Southern California, with temporary headquarters in Parker, Ariz.

JOHN R. MONAGHAN, formerly Chief Engineer for the Del Balso Construction Corporation, of Brooklyn, N.Y., has established a consulting engineering practice at 274 Madison Avenue, New York, N.Y.

G. H. KNUTSON has been elected to the presidency and directorate of the Central Public Utility Corporation (of Delaware), with New York offices at 90 Broad Street.

JAMES C. JORDAN, previously engineer of bridge records, Bureau of Bridges, Department of Public Works, Allegheny County, Pennsylvania, has been appointed county surveyor for Allegheny County, and engineer of plans and records, Department of Planning.

JOHN E. ELLIOTT has been appointed division engineer of the American Bridge Company in Pittsburgh. He was previously an assistant division engineer of that company.

B. P. THOMAS was recently appointed Chief Engineer of the Inter-County River Improvement Commission, with headquarters in Sumner, Wash.

LEROY A. SWANSON has accepted a connection with the Middle West Utilities Company, of Chicago, Ill. He was previously a traffic student with the American Telephone and Telegraph Company in the same city.

Changes in Membership Grades

Additions, Transfers, Reinstatements, Deaths, and Resignations

From August 10 to September 9, 1933, Inclusive

ADDITIONS TO MEMBERSHIP

ALLEN, TOM JOHNSON (Assoc. M. '33), Civ. Engr. (Allen & Rowe), 309 G St., San Diego, Calif.

ARNOLD, GERALD EUGENE (Assoc. M. '33), Water Purification Engr., San Francisco Water Dept., Millbrae, Calif.

BLISS, JOHN HARVEY (Assoc. M. '33), Engr., State Engrs. Office, Santa Fe, N.Mex.

DIECK, WILLIAM HENRY (Assoc. M. '33), Pres., United Elec. Light & Power Co., 4 Irving Pl., New York (Res., 410 Wilson Ave., Brooklyn), N.Y.

ETTINGER, LOUIS JOSEPH, JR. (Jun. '33), With Onondaga County Highway Dept., Skaneateles, N.Y.

GARRAN, FRANK WARREN (M. '33), Prof., Civ. Eng. and Acting Dean, Thayer School of Civ. Eng., Dartmouth Coll. (Res., 12 Sargent St.), Hanover, N.H.

GLASSMIRE, HARRY MARTIN (M. '33), Asst. City Engr. (Res., 61 East Depew Ave.), Buffalo, N.Y.

GUTHRIE, MARION CULBERTSON (Jun. '33), Asst. Plan Draftsman, Design Dept., State Highway Comm., 2312 West 21st St., Topeka, Kans.

HOLSTEIN, PAUL WHERRITT, JR. (Jun. '33), Draftsman, Div. of Eng. and Constr., City of Columbus (Res., 235 Sixteenth Ave.), Columbus, Ohio.

LAHDE, WALTER (Assoc. M. '33), Senior Engr., Jensen, Bowen & Farrell and H. E. Riggs (Res., 1205 Henry St.), Ann Arbor, Mich.

LYDER, AUDLEY ARNOTT (Jun. '33), 1464 Southern Boulevard, New York, N.Y.

MCLEOD, DONALD EDWARD (Jun. '33), 41 North Fullerton Ave., Montclair, N.J.

MCMANUS, THOMAS KESSLER (Assoc. M. '33), Supt., Puget Sound Bridge & Dredging Co., Box L 1, Avalon, Catalina Island, Calif.

MILLS, BRUCE HOPP (Assoc. M. '33), Civ. Engr., Rancho Sespe, Fillmore, Calif.

PERSONS, DAVID WAITE (Jun. '33), With Mississippi River Comm., Box 665, Vicksburg, Miss.

POLLOCK, HERBERT WILLIAM (Jun. '32), Biltmore Apartments, Olive Way and Summitt, Seattle, Wash.

QUENEAU, ROLABD BLAISDELL (Assoc. M. '33), Field Engr., The Pitometer Co., 50 Church St., New York, N.Y.

SMITH, W. SHERMAN (Assoc. M. '33), Asst. Prof., Civ. Eng., Univ. of Toledo (Res., 1827 Cherry St.), Toledo, Ohio.

SPRAGUE, JOHN HANLY CARROLL (Assoc. M. '33), Insp., U.S. Engrs., Care, U.S. Engr. Office, Marmet, W.Va.

STEELE, JAMES POLK (Assoc. M. '33), Care, Long Beach Constr. Co., 2526 East 11th St., Long Beach, Calif.

STOREY, ARTHUR LIPSCOMB (Jun. '33), Chairman United Gas Public Service Co. (Res., 2116 Baldwin St.), Houston, Tex.

TROXELL, JOHN ADRON (Jun. '33), 1804 East 50th, Seattle, Wash.

URBINO, JACINTO (Assoc. M. '33), Engr. in Chg., Water Works Bureau, City of San Juan (Res., 73 Tapia St.), San Juan, Puerto Rico.

VON WEYMARN, PETER (M. '33), 504 West 131st St., Apartment 5, New York, N.Y.

MEMBERSHIP TRANSFERS

AXLINE, EDWIN JASPER (Jun. '28; Assoc. M. '33), Project Engr., State Highway Comm., Billings, Mont.

BETTERLY, JOHN AUSTIN (Jun. '28; Assoc. M. '33), Field Engr., The Dravo Contr. Co., Pittsburgh (Res., 1147 West Elm St., Scranton), Pa.

CARLSON, ROY WASHINGTON (Jun. '28; Assoc. M. '33), Research Engr., Univ. of California, 303 Eng. Materials Laboratory, Univ. of California (Res., 1059 Euclid Ave.), Berkeley, Calif.

MCCULLOUGH, JAY WHEELOCK (Jun. '22; Assoc. M. '33), Civ. Engr., 838 Majestic Bldg., Denver, Colo.

MATTHEWS, JOHN HORACE (Jun. '27; Assoc. M. '33), Instr., Civ. Eng., Purdue Univ., Lafayette (Res., 147 South Grant St., West Lafayette), Ind.

PANHORST, FREDERICK WILLIAM (Assoc. M. '23; M. '33), Acting Bridge Engr., State Highway Comm. (Res., 2620 Eye St.), Sacramento, Calif.

RICHARDSON, NORMAN DAYTON (Assoc. M. '24; M. '33), 18 Cherwing Rd., Bryn Mawr Knolls, Yonkers, N.Y.

SCHIEBER, OLIVER JAY (Assoc. M. '21; M. '33), Senior Engr., Met. Water Dist. of Southern California, Los Angeles (Res., 1751 Windsor Rd., San Marino), Calif.

WOODS, EDWIN MARECHAL (Jun. '28; Assoc. M. '32), 628 North 32d St., Philadelphia, Pa.

REINSTATEMENTS

FUCIK, JOHN FRANK, JR., Assoc. M., reinstated Sept. 1, 1933.

RESIGNATIONS

GILBERTY, FRANK RALPH, Jun., resigned Aug. 17, 1933.

HORTON, FREEMAN HUDSON, M., resigned Sept. 6, 1933.

RAMSAY, GEORGE CHESTER, Assoc. M., resigned Sept. 6, 1933.

TILTON, CHARLES WILFRID, Jun., resigned Sept. 6, 1933.

DEATHS

FORTIER, SAMUEL. Elected M., Aug. 31, 1915; died Aug. 17, 1933.

GAYLER, CARL. Elected M., Sept. 3, 1884; died Sept. 1, 1933.

HALL, WILLIAM HENRY. Elected Assoc. M., Oct. 4, 1910; M., Nov. 27, 1917; died Aug. 14, 1933.

OTAGAWA, MASAYUKI. Elected M., Jan. 2, 1895; died June 29, 1933.

RICE, JAMES HORNER. Elected Assoc. M., Oct. 8, 1918; died Aug. 8, 1933.

ROURKE, LOUIS KEEGAN. Elected M., July 1, 1909; died Aug. 23, 1933.

SNOW, JONATHAN PARKER. Elected M., June 3, 1885; died Sept. 4, 1933.

SPERRY, HENRY MUHLENBERG. Elected M., Nov. 6, 1901; died Sept. 2, 1933.

STOWELL, CHARLES FREDERICK. Elected M., Apr. 4, 1888; died Aug. 31, 1933.

STUBNER, CHARLES ERNEST. Elected M., Mar. 5, 1928; date of death unknown.

TURNER, ORVILLE HICKMAN BROWNING. Elected M., Feb. 2, 1909; died May 16, 1933.

VANDEVANTER, CHARLES OSCAR. Elected M., Sept. 2, 1896; died Aug. 14, 1933.

TOTAL MEMBERSHIP AS OF SEPTEMBER 9, 1933

Members	5,778
Associate Members	6,278
Corporate Members	12,056
Honorary Members	18
Juniors	2,893
Affiliates	112
Fellows	5
Total	15,084

Men Available

These items are from information furnished by the Engineering Societies Employment Service, with offices in Chicago, New York, and San Francisco. The Service is available to all members of the contributing societies. A complete statement of the procedure, the location of offices, and the fee is to be found on page 97 of the 1933 Year Book of the Society. To expedite publication, notices should be sent direct to the Employment Service, 31 West 39th Street, New York, N.Y. Employers should address replies to the key number, care of the New York Office, unless the word Chicago or San Francisco follows the key number, when the reply should be sent to the office designated.

CONSTRUCTION

CONSTRUCTION ENGINEER; Assoc. M. Am. Soc. C.E.; 46; single; 22 years experience in field engineering and field supervision of construction work, including earth, timber, and concrete dams, hydro-electric and steam stations, sewage beds, manufacturing plants, composite wharf, golf course, recreation park, and research laboratories. Available immediately. D-2513.

CONCRETE TECHNICIAN; Assoc. M. Am. Soc. C.E.; civil engineering degree, University of West Virginia, 1925; married; 8 years construction experience, specializing in concrete. Especially qualified to take charge of concrete field laboratory and concrete inspection force, supervising testing, designing mixtures, and control of field concrete. Location immaterial. Available on notice (two weeks to one month). C-8022.

DESIGN

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 28; married; civil engineer's degree, Polytechnic Institute of Brooklyn; special course in structural engineering; 9½ years practical experience in structural steel design for subways, piers, pier sheds, tunnels, buildings in railroad yards, and elevated structures. Checker of shop drawings; excellent draftsman. Available immediately. Location anywhere. D-1754.

CIVIL AND INDUSTRIAL ENGINEER; M. Am. Soc. C.E.; graduate; licensed engineer and architect. Design and construction of industrial plants, commercial garages, housing groups, institutional developments, and hydro-electric power houses. In complete charge of projects, including preliminary investigations, complete layouts, detailed designs, specifications, letting contracts, supervising all construction operations, etc. B-2835.

STRUCTURAL ENGINEER; Jun. Am. Soc. C.E.; 27; B.S. (C.E.); married; one child; 6 years experience: chiefly in steel and concrete design, steel detailing, and checking; miscellaneous experience in surveying and machine design. Location anywhere in the United States; prefer job as chief draftsman with small steel company. Reasonable salary. Available two weeks notice. D-2452.

CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 33; licensed engineer, New York State; married; 8½ years experience in steel and reinforced concrete design; estimating; checking steel designs and shop drawings; surveying; and superintendent of construction. Would consider position as structural draftsman. Location New York State. Available immediately. D-2462.

STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 37; married; graduate; state licenses; 10 years experience, surveying, design, and construction. Design, detail, and preparation of plans and specifications, on railroad, highway, parkway, harbor, and tall building structures. Completely familiar with the design of rigid-frame bridges. Available immediately. D-1496.

DESIGNING AND CONSTRUCTION ENGINEER; M. Am. Soc. C.E.; university graduate; state licenses; with 30 years experience on investigations, design, and construction of dams, major irrigation structures, and hydro-electric plants, and on reports for the development of watersheds for power, navigation, irrigation, and flood control; desires assignment. Speaks Spanish. Available immediately. B-2815.

ENGINEER; Assoc. M. Am. Soc. C.E.; B.S. in C.E.; 32; married; 3 years instructor of drafting and shop work; 3 years highway location, construction, drafting, computing, bridge inspection; 3½ years structural designer with oil company. Registered professional engineer in Pennsylvania. Speaks fair Spanish. Will seriously consider anything offered. C-5010.

EXECUTIVE

OFFICE ENGINEER-ARCHITECT; M. Am. Soc. C.E.; Am. Inst. Arch.; 39; married; American; Cornell University; 12 years structural and architectural design, drafting, specifications, detailed estimating, and supervision, on residences, churches, hospitals, schools, and commercial buildings; 5 years projection, design of structures, maps, profiles, and estimates on highway and railway location; experience in United States and Latin-America. Fluent Spanish, including technical vocabulary. D-2472.

AVAILABLE IMMEDIATELY; Assoc. M. Am. Soc. C.E.; very capable manager engineering and construction; able to organize, equip, and administer largest engineering and construction projects; graduate civil engineer; 24 years experience hydro-electric irrigation investigations, designs, construction, railroad mining, and mine plant construction; 9 years foreign experience in complete charge of heavy construction for largest industrial and contracting companies. Salary open. D-2453.

CIVIL ENGINEER; M. Am. Soc. C.E.; 20 years experience in office and field, covering surveys, construction, design, estimating, drafting, research, and reports. Has been connected with large railroad companies and other organizations including important flood-prevention project. Continuously employed until recent times. Will accept moderate salary. B-9037.

CONSTRUCTION ENGINEER; Assoc. M. Am. Soc. C.E.; 36; married; graduate of Rutgers University; New York State license; desires to locate with contractor; 12 years experience on subways, water supply tunnels, and dams. Location immaterial. Available immediately. A-3625.

GRADUATE CIVIL ENGINEER; Assoc. M. Am. Soc. C.E.; 33; New Jersey license; 14 years experience, including harbor work on surveys, general maintenance, collecting data, and preparing plans for extensive development; design of various types of steel and reinforced concrete bridges. Salary open. Location immaterial. Available immediately. D-2493.

JUNIOR

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 24; single; B.S. in C.E., Lehigh University, 1932; 1½ years with municipal engineer, surveying, drafting, computing, and inspection. Desires position in any branch of civil engineering. Location immaterial. Available immediately. D-2036.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 27; single; B.S. in C.E., Rutgers University, 1930; 1 and ⅔ years transitman, Essex County Highway Department; passed New Jersey State Civil Service Senior Draftsman examination. Desires working or teaching position in any branch of civil engineering, preferably one involving mathematics. D-663.

JUNIOR ENGINEER; Jun. Am. Soc. C.E.; 25; B.S. in C.E.; member of Sigma Tau Fraternity; 3 years experience in stream gaging, construction and operation of water-stage recorder installations, analysis of stream-flow data, and preparation of reports for publication. Available immediately. D-2458.

ENGINEER; Jun. Am. Soc. C.E.; 25; single; B.S. in A.E., Pennsylvania State College, 1930; experience with leading Eastern steel fabricator, shop and office, on time-study and cost-data work, group bonus plans, detailing, estimating, and general office work; timekeeper and assistant engineer on erection of various types of steel buildings. Good references. Available immediately. Any location. Reasonable salary. D-1016.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 25; graduate of Worcester Polytechnic Institute, 1932; 1½ years experience, surveying, drafting, computing on highways, parks, and parkways. Wants position in any branch of civil engineering. Willing worker. References. Location immaterial. C-6231.

ENGINEER; Jun. Am. Soc. C.E.; 23; single; B.S. in C.E.; experience with the Pennsylvania Department of Highways as inspector; gas service inspector for the Philadelphia Gas Works; experience on pneumatic caisson foundations; with the Pennsylvania Railroad on electrification from New York to Philadelphia; and some experience in automotive work. Location immaterial. D-2295.

CIVIL ENGINEER; Student Member; 22; married; B.S. in C.E.; Case School of Applied Science, 1933; 2 months experience in concrete dam construction; majored in water supply design and construction; experienced in publicity and newspaper work; desires legitimate employment of any sort. Available immediately. Location immaterial. D-2521.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; graduate, 1930; neat, industrious, ambitious, resourceful. Speaks Italian, Spanish, English. Good mathematician. Competent draftsman, 4 years experience. Capable of performing and supervising difficult drafting and computations for construction drawings, highways, sewers, pipe lines; 2½ years experience on highways. Desires work with reputable concern, consulting engineer, or as assistant in university. C-7279.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 25; single; B.S. in C.E., Lehigh University, 1931; 3 months as draftsman; 1 month surveying experience; 8 months as sales engineer and welding-specification writer. Resourceful, dependable. Desires engineering or teaching position. Salary and location open. Available immediately. C-8904.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 26; single; B.S. in C.E., 1930; 2 years experience in highway engineering, including preliminary surveys, preparation of plans, calculating quantities, and construction with N.J. State Highway Department. Passed U.S. Civil Service Examination for Junior Civil Engineer. Desires opportunity in any branch of civil engineering. Available at once. C-8158.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 30; with college and technical degrees. American with absolute command of Spanish and knowledge of French. General experience, especially in steel design and reinforced concrete power and oil house work. Used to handling common labor. Will go anywhere, preferably Spain or South America. C-4088.

CIVIL ENGINEER; Jun. Am. Soc. C.E.; 23; single; B.S. in C.E., in structural, hydraulic, and general C.E. options, University of Illinois, 1932. Member Tau Beta Pi; associate Sigma Xi. Summer experience on survey corps. Good draftsman. Best references. Speaks German. Location immaterial in United States or foreign countries. D-2504.

CIVIL AND STRUCTURAL ENGINEER; Jun. Am. Soc. C.E.; 29; single; B.S. in C.E., 1927; 4 years experience with large Eastern steel company in detailing, checking, and layout work in the engineering department; 1 year experience in field and office with land surveyor, 2 years design work on steel and concrete highway bridges and viaducts. Prefers position with consulting engineer or architect. D-2503.

SANITARY ENGINEER; Jun. Am. Soc. C.E.; single; 24; B.S. in C.E., 1931, University of California; 2 years sanitary engineer (laboratory and field work) for seven water utilities in the Northwest. Some experience in sewage disposal plant. Desires position with consulting, sanitary, or irrigation engineer; state or city health department. Available on short notice. D-2514.

JUNIOR ENGINEER; Jun. Am. Soc. C.E.; 29; technical education; 6½ years engineering experience as follows: 3 years municipal; 2 years highway; 1½ years mining in South America. Experienced running party on topographic, route, and underground surveys. Speaking and writing knowledge of Spanish. Accept any desirable domestic or foreign position. D-2515.

SALES

FOREIGN SALES-BUILDINGS AND ROADS; Assoc. M. Am. Soc. C.E.; 45; 8 years experience selling in British Empire, principally India, but including six months in Africa; acquainted London and Egypt. Lately independent factory representative, Bombay; 2 years early railroad experience, South America. Some knowledge of Spanish; read French. Education, Andover and Harvard. B-6364.

SALES MANAGER AND STRUCTURAL ENGINEER; Assoc. M. Am. Soc. C.E.; 29; married; graduate; high-class sales executive with long experience in sales and structural engineering, specializing in steel requirements for bridges and buildings. Seeking connection with building material manufacturer or general contractor. Qualified district sales manager. Good designer and estimator. Available soon. D-16.

STRUCTURAL ENGINEER; M. Am. Soc. C.E.; 46; married; extensive successful experience in engineering and sales of structural steel; looking for a responsible position. Desires opportunity to give detailed information. C-5095.

TEACHING

M. I. T. GRADUATE IN CIVIL ENGINEERING; Jun. Am. Soc. C.E.; single; with 4 years experience teaching differential and integral calculus and 6 years experience in all phases of building industry; desires full-time teaching position or fellowship for year 1933–1934 in mathematics or civil engineering. Location immaterial. C-3262.

ENGINEER; Assoc. M. Am. Soc. C.E.; 15 years experience on location, design, and construction, mostly on work appurtenant to water supply. B.C.E. and M.S. degrees from a leading university. Would like to teach municipal engineering and/or surveying, preferably in the Middle West. D-2491.

RECENT BOOKS

New books of interest to Civil Engineers, donated by the publishers to the Engineering Societies Library or to the Society's Reading Room, will be found listed here. A comprehensive statement regarding the service which the Library makes available to members is to be found on page 87 of the Year Book for 1933. These notes regarding the books are taken from the books themselves, and this Society is not responsible for them.

BILDWORT ENGLISCH TECHNISCHE SPRACHHEFTE 7, CIVIL ENGINEERING, Berlin, VDI-Verlag, 1933. 32 pp., diagrs., 8 × 6 in., paper, 1.50 rm.

A pamphlet to assist the German-speaking engineer to a knowledge of technical English in the field of civil engineering. By means of drawings, in which each element is marked by its English name, and a text on building materials, bridges, highway and hydraulic engineering, etc., a large vocabulary is made available.

DIE WÄRMEÜBERTRAGUNG AN KALTE FLÄCHEN BEI FREIER STRÖMUNG. (BEIHEFTE ZUM GESUNDHEITS-INGENIEUR, Reihe 1, Heft 31.) By W. Piening. Munich and Berlin, R. Oldenbourg, 1933. 23 pp., diagrs., charts, tables, 12 × 9 in., paper, 4.40 rm.

An investigation of the transfer of heat to cold plates at low room temperatures, together with the influence of the moisture of the air on the rate of transfer. The coefficient of transfer under these conditions was obtained and the effect of moisture determined.

DER WIRTSCHAFTLICHE AUSBAU DER GROSSSTADT. By K. Stodieck. Berlin, VDI-Verlag, 1933. 59 pp., diagrs., charts, tables, 8 × 6 in., paper, 3 rm.

This book presents a theoretical plan for large cities in which industrial and business districts are distinctly separated from the residential districts. Using Berlin as an example, the author shows how it might be reconstructed with large economic and social benefits.

ENGINEER'S MANUAL OF ENGLISH. By W. O. Sypherd and S. Brown. Chicago, Atlanta, Dallas, and New York, Scott, Foresman and Co., 1933. 515 pp., charts, diagrs., tables, 7 × 5 in., leather, $2.

The first half of this work is a concise textbook of English composition, in which emphasis is placed on correspondence, reports, articles, bulletins, specifications, and other forms of writing for engineers. The second contains specimens of engineering writing for study and analysis. The work is also valuable for reference.

FORSCHUNGSHEFT 358. TECHNISCHE GESTEINPRÜFUNG. By H. Burchartz, G. Saenger, and K. Stäcke. Berlin, VDI-Verlag, 1933. 23 pp., illus., diagrs., charts, tables, 12 × 8 in., paper, 5 rm.

A report on an elaborate investigation of methods of testing stone. The points investigated were the relations between the petrographic structure of stone and its mechanical properties, and the usefulness of the various accepted methods of testing.

HEROIC AGE OF SCIENCE—THE CONCEPTION, IDEALS, AND METHODS OF SCIENCE AMONG THE ANCIENT GREEKS. By S. A. Heidel. Published for Carnegie Institution of Washington by Williams and Wilkins Co., Baltimore, 1933. 203 pp., 9 × 6 in., cloth, $2.50.

This book exhibits by examples the way in which the Greeks thought of science and tried to solve its problems, and points out the relation between their procedure and the processes of the mind in practical affairs. The original connection of Greek science with magic and religion is traced, and efforts toward emancipation from ancient beliefs are described.

IGNEOUS ROCKS AND THE DEPTHS OF THE EARTH. By R. A. Daly. New York and London, McGraw-Hill Book Co., 1933. 598 pp., illus., diagrs., charts, tables, 9 × 6 in., cloth, $5.

A consideration of the leading facts known about eruptive rocks, particularly the facts concerning field relations. The second part gives a general eclectic "theory of the earth," and the third part sketches the application of this theory to the separate major groups of eruptive rocks.

PLANNING FOR THE SMALL AMERICAN CITY (Publication 32). By R. V. Black. Chicago, Public Administration Service, 1933. 90 pp., illus., diagrs., maps, tables, 11 × 8 in., paper, $1.

This monograph considers planning in cities with a population under fifty thousand, from the viewpoint of operating officials and interested citizens. How a plan may be made, what planning offers a city, and how to carry out a plan are discussed; and sound advice, based upon recent expert opinion, is given.

REINFORCED CONCRETE-MECHANICS AND DESIGN. By R. A. Caughey. Ann Arbor, Mich., Edwards Bros., 1933. 115 pp., diagrs., charts, tables, 11 × 8 in., paper, $2.70.

The aim of this text is to assist the undergraduate student to the mastery of fundamental principles, which are necessary both in routine design and the solution of unusual problems. The chapters discuss such subjects as: web stresses in beams; the design of beams; columns; combined bending and direct stress; moments in beams and frames; floors carrying concentrated loads; composite members of concrete and structural steel; and the design of buildings, arches, rigid frames, dams, and retaining walls.

STOP THAT SMOKE! By H. Obermeyer. New York and London, Harper and Brothers, 1933. 289 pp., illus., 9 × 6 in., cloth, $2.50.

A popular, non-technical discussion of the smoke nuisance and means for its abatement, from the viewpoints of health, civic beauty, property damage, and fuel economy. The methods of education, legislation, and regulation that have been used are described. It affords an excellent review of the methods adopted in American cities and the results achieved.

VERSUCHE ZUR BESTIMMUNG DES TANGENTIALEN SOHLENWIDERSTANDES VON GEWICHTSSTAUMAUERN. By N. Kelen. Berlin, Selbstverlag des Verfassers, im Vertrieb: Hirschwaldsche Buchhandlung, 1933. 42 pp., illus., diagrs., charts, tables, 10 × 6 in., paper, 4 rm.

As most dam failures occur through sliding, the frictional resistance between concrete and rock is important. By means of models, the author of this pamphlet has determined the effect of various factors upon frictional resistance, and drawn conclusions for practical use in design. The work describes the apparatus and method of investigation and gives the results.

CURRENT PERIODICAL LITERATURE

Abstracts of Articles on Civil Engineering Subjects from Magazines in This Country and in Foreign Lands

Selected items from the current Civil Engineering Group of the Engineering Index Service, 29 West 39th Street, New York, N.Y. Every article indexed is on file in The Engineering Societies Library, one of the leading technical libraries of the world. Some 2,000 technical publications from 40 countries in 20 languages are received by the Library and are read, abstracted, and indexed by trained engineers. With the information given in the items which follow, you may obtain the article from your own file, from your local library, or direct from the publisher. Photoprints will be supplied by this library at the cost of reproduction, 25 cents per page, plus postage, or technical translations of the complete text may be obtained at cost.

BRIDGES

CONCRETE ARCH, GREAT BRITAIN. Three Reinforced Concrete Bridges Over River Thames Opened by H. R. H., the Prince of Wales, July 1933. *Concrete and Constr. Eng.* (*supplement*), vol. 28, no. 7, July 1933, supp. pp. 1–49. Concrete for Bridges, A. Dryland; Modern Bridge Design, M. Ayrton; Twickenham Bridge, W. L. Scott; Hampton Court Bridge, T. J. Gueritte; Chiswick Bridge, W. L. Scott; Progress in Reinforced Concrete Bridge Construction.

CONCRETE GIRDER, WASHINGTON, D.C. Small Concrete Bridge Simulates a Timber Structure. D. H. Gillette. *Eng. News-Rec.*, vol. 111, no. 6, Aug. 3, 1933, pp. 140 and 141. Design and construction of the Lyons Mill Bridge in Rock Creek Park, Washington, D.C., consisting of 3-span continuous T-beam girders; total length 110 ft, with corbeled girders, hand-rails stained with creosote, and iron fittings.

EGYPT. Khedive Ismail Bridge, Cairo, Egypt. *Engineering*, vol. 136, no. 3525, Aug. 4, 1933, pp. 105–107, 2 supp. plates. New bridge is 382.2 m long between centers of end-bearings, and consists of two shore spans and four fixed spans; the swing span, with its approach cantilevers, provides two 20-m openings for river craft; superstructure consists of four lines of steel girders placed 4.75 m apart; swing span is carried at 16 points of support in circular girder of compound type.

IRELAND. Reconstruction of Butt Bridge, Dublin, F. W. Bond. *Inst. Civ. Engrs. Ireland—Trans.*, vol. 59, ninety-seventh session, Nov. 1932 to May 1933, pp. 35–98 (discussion) 99–116, 6 supp. plates. Construction of a concrete-arch highway bridge consisting of one span 112 ft long and two spans 40 ft each; roadway is 40 ft wide; details of expansion joints; unit costs.

ONTARIO. Ontario Department of Northern Development Builds Large Highway Bridge at Englehart. *Eng. and Contract Rec.* (formerly *Contract Rec.*), vol. 47, no. 27, July 5 1933, pp. 651 and 654. Construction, in 4 months, of a highway bridge comprising five deck-truss spans; total length 800 ft; maximum span 160 ft long; concreting and steel erection; river protection.

PIERS, FOUNDATIONS. Bombing Out Subaqueous Rock at Golden Gate Bridge Pier. *Eng. News-Rec.*, vol. 111, no. 4, July 27, 1933, pp. 93–95. Small bombs dropped against rock prepare a 15-ft hole for 250-lb bombs; holes spaced 20 ft apart; shattered rock removed by clamshell derrick on barge; extensive excavation by pneumatic process was eliminated.

PLATE GIRDER, IRELAND. Craigavon Bridge, Londonderry. *Engineer*, vol. 156, no. 4045, July 21, 1933, p. 72. Bridge replaces Carlisle Bridge, wrought-iron structure of 11 spans; new bridge is similar in general arrangement and consists of 5 main spans and 17 approach spans; main spans, each of 130 ft, are formed of main girders of the N-type resting on forged-steel rocker bearings.

PONTOON, AFRICA. Floating Rail and Highway Bridge, A. L. Helwig. *Military Engr.*, vol. 25, no. 142, July–Aug. 1933, pp. 309 and 310. Design and construction of a pontoon bridge 1,185 ft long over the Ebrie Lagoon at Abidjan, Ivory Coast Colony, in Africa; the roadway is 8.5 m wide, including tracks of a meter-gage railway using a 39-ton locomotive.

RUSSIA. Russia Builds Rail-Bridge Over Dnieper River, C. L. Christensen. *Eng. News-Rec.*, vol. 111, no. 6, Aug. 10, 1933, pp. 160 and 161. Construction of single-track railroad bridge at Dniepropetrovsk; total length 5,350 ft, consisting of fourteen 170-ft arches, fourteen 95-ft arches, three 50-ft arches, four 53-ft continuous-girder spans, and two 360-ft steel through trusses.

STEEL, STRESSES. Impact Effect of Live Loads Upon Bridges, C. F. Koerner. *Inst. Engrs. Australia—Journal*, vol. 5, no. 6, June 1933, pp. 181–192. Theoretical study of causes of impact; impact due to rapidity of application of load; impact due to periodically varying forces whose period coincides with the natural vibration period of structure; recommended methods for determining impact on the superstructure; impact on substructure; vibration period of uniform girder and of uniform girder carrying single concentrated load.

STEEL TRUSS. Nipawin Bridge, A. R. Ketterson. *Eng. Journal*, vol. 16, no. 6, June 1933, pp. 252–259. Design and construction of combined steel-truss bridge over the Saskatchewan River; 1,908 ft total length; comprising four 280-ft truss spans which were erected by cantilevering.

STRUCTURAL STEEL, WELDING. Erecting Steel Buildings and Strengthening Steel Bridges by Welding—II, F. P. McKibben. *Am. Welding Soc.—Journal*, vol. 12, no. 6, June 1933, pp. 4–14. Strengthening wrought iron and steel bridges by welding; methods of repairing metallic bridges; charts showing some typical welded bridge details; comparative loading tests of welded, riveted, and bolted floor panels made of channels; writer's recommended tests for qualifying operators engaged in welding bridges or buildings; flame cutting.

SUSPENSION, WASHINGTON. Short-Span Suspension Bridge Uses Prestressed Rope Cables, H. O. Blair. *Eng. News-Rec.*, vol. 111, no. 3, July 20, 1933, pp. 70 and 71. Cable arrangement, tower top, and anchorage details, stiffening truss, and roadway design of the new 300-ft crossing of the Lewis River near Yale, Wash.

TRESTLE. Trestle Design to Resist Rio Grande Floods, J. W. Beretta. *Eng. News-Rec.*, vol. 110, no. 25, June 22, 1933, pp. 802–804. Construction of 2,000 ft of American approach to the international bridge at Del Rio, Tex.; trestle pile bents are braced by concrete sill and galvanized cable diagonals.

VIADUCTS, LIGHTING. Lighting New Sixth Street Viaduct, L. L. Huot. *Western City*, vol. 9, no. 7, July 1933, pp. 19 and 20. Lighting of viaduct in Los Angeles; specially designed Marbelite standards carry lanterns and also afford a support for any future trolley-wire installation; 156 large lanterns used; 400-ch lamps installed.

BUILDINGS

EXHIBITION BUILDINGS, CHICAGO. Engineering Research and Building Construction, B. M. Thorud. *Arch. Forum*, vol. 59, no. 1, July 1933, pp. 65–69. Review of principles used in the planning and construction of buildings of Century of Progress Exposition in Chicago; building codes; foundations; 2-story buildings; basis of selection of materials; wall criteria; exterior wallboard on studs; interior wallboard on studs; wearing surfaces; roofs; floors; framing.

RADIO CENTER, NEW YORK CITY. Rockefeller Center in New York City, R. Fleming. *Engineering*, vol. 136, no. 3523, July 21, 1933, pp. 53–55. Particulars concerning the largest commercial building project ever undertaken by private capital.

CITY AND REGIONAL PLANNING

TENNESSEE. Tennessee Valley Authority Act, 1933. *Engineering*, vol. 136, no. 3524, July 28, 1933, pp. 82 and 83. Act envisages active Federal coordinated development on an unprecedented scale, which, if successful, will be applied to other great areas in the country; under this act the commission has been appointed to develop the valley from the point of view of power production, flood control, navigation, afforestation, prevention of soil erosion, decentralization of industry, etc.

CONCRETE

CONSTRUCTION. Need of Better Concrete, A. M. Bouillon. *Military Engr.*, vol. 25, no. 142, July–Aug., 1933, pp. 319–324. Making concrete impervious to water; concrete piles cast in place; compression and other tests; patching defective spots; protection against destructive agents; injuries from drying; damage from freezing; attacks by chemicals; destructive effect of fire; resistance to earthquakes; unstable structures. (Concluded.)

CURING. Ohms of Resistance Measure Concrete Curing, S. B. Slack. *Eng. News-Rec.*, vol. 111, no. 6, Aug. 10, 1933, pp. 169 and 170. Tests of the cement-spray method of curing concrete pavement by determining its electrical resistance; electrical-resistance curves obtained on concrete pavement cured by three methods.

MIXING. Aggregate Grading and Concrete Quality, H. N. Walsh. *Inst. Civ. Engrs. Ireland—Trans.*, vol. 59, ninety-seventh session, Nov. 1932 to May 1933, pp. 275–338 (discussion) 339–358. Review of the theories of concrete proportioning, with special reference to workability and density; sets of gradings as standards for classifying samples of aggregates; author's grading limits; classifying aggregate samples; designing mixes. Bibliography.

TAMPING. Compaction of Concrete Through Use of Vibratory Tampers, R. E. Davis and H. E. Davis. *Am. Concrete Inst.—Journal*, vol. 4, no. 9, June 1933, pp. 365–372. Investigations in progress in the Engineering Materials Laboratory of the University of California to determine relative advantages of vibratory tamping compared with hand tamping, including tests of ease of placement, form pressures, homogeneity of concrete, bond of new concrete to old, strength, density, durability, and volume changes.

VIBRATING. Vibrated Concrete, T. C. Powers. *Am. Concrete Journal*, vol. 4, no. 9, June 1933 pp. 373–381. Field and laboratory studies made by the Portland Cement Association, of the effect of vibration on concrete; relation of strength to paste quality; relation between compressive strength and cement-voids ratio for normally vibrated concrete; freezing and thawing tests; relation between modulus of rupture and cement-voids ratio for vibrated and hand-placed concrete; possibilities of vibration.

DAMS

AUSTRALIA. Mount Bold Storage, Adelaide Municipal Water Supply, H. J. O. Eaton and T. A. Farrent. *Inst. Engrs. Australia—Journal*, vol. 5, no. 6, June 1933, p. 192. Authors' reply to discussion of their paper previously indexed from issue of Feb. 1933.

BOULDER DAM PROJECT. Boulder Canyon Project, W. R. Young. *Military Engr.*, vol. 25, no. 142, July–Aug. 1933, pp. 296–300. General features of project described in many previously indexed articles; location and capacity of reservoir; main features of design; program of construction.

BUTTRESS. Articulated Buttress-Type Dams for Municipal Water Supply Projects, D. A. Dedel. *Pub. Works*, vol. 64, no. 7, July 1933, pp. 35–37. Recent trends in buttress dam design; fundamental principles; modified buttress type; water-bearing members; dual purpose dams; water-bearing members of various types of dams.

CONCRETE ARCH, WASHINGTON. Diablo Constant-Angle Arch Dam, L. Jorgensen. *Engineering*, vol. 136, no. 3521, July 7, 1933, pp. 2–6. Dam is a typical example of a high constant-angle arch dam spanning a rather large gap; it is the largest of 48 dams of this type in various parts of world; stresses in horizontal arches below and above elevation 960; derivation of coefficients.

CONCRETE GRAVITY, GERMANY. Die Bleiloch-sperre bei Saalburg in Thueringen, H. Hammerstaedt. *VDI Zeit*, vol. 77, no. 27, July 8, 1933, pp. 739 and 740. Description of Bleiloch curved concrete gravity dam—65 m high, 205.5 m maximum length—located near Saalburg in Thuringia, Germany; details of dam appurtenances.

DESIGN. International Congress on Large Dams. *Engineer*, vol. 156, nos. 4045 and 4046, July 21, 1933, pp. 64 and 65; July 28, pp. 90

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VAST ROAD-BUILDING and other construction projects, involving expenditures of hundreds of millions of dollars, are being started by Federal and State Governments.

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THOROUGH CUTS

WHEN a cut is to be made through a hill leaving a bank or wall on either side, use Du Pont Quarry Gelatin, Red Cross Extra, Red Cross Blasting Free-Running Powders or R. R. P. The explosive to select depends upon the nature of the rock and the working conditions.

Quarry Gelatin is made especially for wet outside work. Use the higher strengths for hard rock, and the lower ones for soft or easier-breaking rocks.

If the holes are not particularly moist, Red Cross Extra will give good results. For deep holes in fairly dry work, the Free-Running Red Cross Blasting Powders are very economical.

BOULDERS

FOR mudcapping, remove the dynamite from the shell, pack it in a conical heap on the boulder; insert cap and fuse, cover explosive with several inches of thick, heavy mud. Never lay stones on top of mudcap.

For snake-holing, punch hole beneath boulder and in such a location as to ensure charge being placed against boulder. Tamp charge compactly.

Red Cross Extra 20% or 40%, Du Pont Extra D, or Agritol are effective for snake-holing where there is heavy soil under boulders to provide the required resistance.

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SIDE HILL CUTS

IF in hard rock, use Quarry Gelatins. Softer materials may be successfully handled by Red Cross Extra grades, or in dry work Free-Running Red Cross Blasting or granular black powder may be used.

In working from the side, slight variations are made, depending on whether excavated material is to be used for filling or is wasted down the hill. If it is to be used for filling, the loading should be barely heavy enough to break the ground for convenient handling. In working from the end, the rules for thorough cuts apply, and the same explosives are recommended.

STUMPS

STUMPS in firm soil are more easily blasted than when located in sandy soils. For blasting green, lateral rooted stumps, use 40% Red Cross. For tap-rooted stumps, use Agritol, or, if soil is heavy, Red Cross Extra 20%; if light soil, use Red Cross Extra 40%. To blast tap-rooted stumps out of light soil, use Red Cross Extra 40%.

For blasting the big stumps in the Pacific Northwestern States, use Du Pont Loggers' Powder.

QUARRYING

IF stone is to be crushed for road building, use Red Cross 40%, Du Pont Extra, Gelatin, or Gelex. Holes should be well tamped and charges fired simultaneously.

For quarrying dimension stone, use blasting powder of fine granulation to start cracks and seams in the desired direction. For extremely hard rock of the granite or trap types, Du Pont Quarry Gelatin should be used.

GRAVEL PITS

BLASTING is effective for speeding up excavation to obtain grading material. Bore holes are spaced about as for other blasting. If rock is not encountered, holes are loaded much lighter — the object being to loosen the material sufficiently to make digging easy. Use Red Cross Extra 20% and Red Cross Blasting No. 2 F. R. for this work.

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FILL SETTLEMENT

DYNAMITE can be effectively employed for removing muck and other unstable material from roadbeds. Dynamite is exploded to create cavities for the fill to drop into, and also to stir up and liquefy the mud surrounding the cavity to permit the rapid settlement of the fill.

Du Pont Ditching Dynamite is particularly effective, because of its water-resisting and propagating qualities.

If necessary to place the explosive under the fill, use Du Pont 40% Gelatin in large-size cartridges.

DITCHING

DITCHES can be blasted in wet soil by the propagation method; or the electric method can be used in wet or dry soil.

In wet soil, the propagation method, when used with Du Pont Ditching Dynamite, simplifies drainage construction and effects economies in time, labor and money. Ditching with dynamite is frequently successful where conditions make other methods impractical.

and 91, and editorial comment pp. 89 and 90. Account of meeting held in Sweden and Norway during June and July 1933; four questions selected for discussion: deterioration by aging in concrete of gravity dams; influence of internal temperature and distortion of gravity dams; research methods to ascertain whether a given material is suitable for construction of an earth dam; physical laws governing infiltration of water through dam and subjacent soil.

HYDRAULIC FILL, CALIFORNIA. Non-Slip Hydraulic-Fill Dam for San Diego, H. N. Savage. *Eng. News-Rec.*, vol. 111, no. 2, July 1933, pp. 33–36. Design and construction of the El Capitan Dam on the San Diego River, for the water supply of San Diego, Calif.; structure is hydraulic-fill with rock embankment 217 ft high, cut-off wall 53 ft deep, length 1,200 ft, and side slopes varying from 1:15 to 1:3; slip and spread are restrained by concrete toewalls and heavy armoring of riprap; downstream drainage tunnels.

HYDRAULIC GATES, SPECIFICATIONS. Butterfly Valves for Boulder Power Plant. *U.S. Bureau of Reclamation—Specifications*, no. 544, 1933, 41 pp., 84 supp. plates. Invitation for bids, schedules, specifications, and drawings of butterfly valves for the Boulder power plant, Boulder Canyon project.

MARYLAND. Redesign and Construction of Prettyboy Dam. *Eng. News-Rec.*, vol. 111, no. 3, July 20, 1933, pp. 63–67. Redesign necessary as a result of the discovery of rotten rock in the foundation of the Baltimore water supply dam; dam as constructed is about 167 ft high and 853 ft long; river control; use of wire saws; concrete plant; grouting; power plant.

FLOOD CONTROL

CALIFORNIA. Analysis of Flood Produced by Severe California Cloudburst. *Eng. News-Rec.*, vol. 111, no. 6, Aug. 3, 1933, p. 128. Description of flood from cloudburst precipitation of 4.34 in. in 5 hr, in the Tehachapi Valley, southern California; flood discharge at peak is estimated up to 500 cu ft per sec per sq mile.

RIVERS, IMPROVEMENT. New Plans for Mississippi. Developing Atchafalaya for Greater Floodflow. *Eng. News-Rec.*, vol. 111, no. 4, July 27, 1933, pp. 102–105. Creating a self-maintaining channel in the Atchafalaya River by shortening, slope equalization, and the removal of constrictions by cut-offs and agitation dredging.

UNITED STATES. Record Floods in Colorado, Wyoming, and Arizona Formulized, P. V. Hodges. *Eng. News-Rec.*, vol. 111, no. 6, Aug. 10, 1933, p. 171. Flood formulas for various river basins; comparison of flood formulas of principal streams in Colorado, Wyoming, and Arizona; prediction of future floods.

FLOW OF FLUIDS

FLOW OF WATER, VORTEXES. Free Vortexes, E. B. Ball. *Engineer*, vol. 155, no. 4042, June 30, 1933, p. 654. Illustrations of vortex formed at Loch Treig when lake was discharging through pressure tunnel and of another vortex taken in the reservoir of the Arapuni power scheme, in New Zealand, formed when the reservoir was being emptied through a diversion tunnel; observations made at a number of reservoir outlets revealed that a free vortex escapes the whirl, formed at the surface of the reservoir, and rotates in a counter-clockwise direction in the Northern Hemisphere and clockwise in the Southern Hemisphere.

HYDRAULIC ENGINEERING

FLUMES. Improved Transition Adds to Flume Capacity, J. A. Fraps. *Eng. News-Rec.*, vol. 111, no. 4, July 27, 1933, pp. 110–112. Change in hydraulic characteristics of transition, using timber lining, increases flow limit from 360 to 400 cu ft per sec in the Agua Fria River flume of the Roosevelt Irrigation District.

HYDRAULIC LABORATORIES, UNITED STATES. Hydraulic Laboratories in United States. *U.S. Bureau of Standards—Reference:* YI-611NH-655-C., Washington, D.C., July 1, 1933, 78 pp. Data on 63 hydraulic laboratories operated by universities, Federal and state experiment stations, and private companies.

HYDRAULIC MODELS. Experiments with Movable Bed River Models, H. D. Vogel. *Military Engr.*, vol. 25, no. 142, July-Aug. 1933, pp. 313–316. Report on hydraulic laboratory study, by the U.S. Waterways Experiment Station, of Island Number 9 and Race Track Towhead reaches of the Mississippi River; effects of dredging; effects of dikes.

HYDROLOGY, METEOROLOGY, AND SEISMOGRAPHY

WATER SUPPLY, GEOLOGY. Hydro-Geological Surveys, E. Morton. *Engineer*, vol. 156, no. 4046, July 28, 1933, p. 94. Demonstration of the principle that successful cooperation by rural authorities, both on engineering and administrative sides, involves as a primary factor the local hydro-geological survey of water resources of rural areas. Before Brit. Water Works Assn.

INLAND WATERWAYS

RIVERS. Lower Danube—Plan for Harnessing Iron Gates. *Times Trade and Eng. Supp.*, vol. 32, no. 786, July 29, 1933, p. 448. Review of the report issued by the Roumanian National Institute for Study of Harnessing and Utilization of Sources of Energy on the canalization of the Iron Gates section of the river and the development of hydro-electric energy at dams, which would be required to improve the conditions of navigation; scheme would enlarge the capacity of the waterway almost indefinitely by doing away with rapid currents; two dams suggested.

LAND RECLAMATION AND DRAINAGE

CULVERTS. Place and Merits of Corrugated Metal Culverts. *Eng. and Contract Rec.* (formerly *Contract Rec.*), vol. 47, no. 27, July 5, 1933, pp. 656 and 657. Proper size of culverts; design; merits of corrugated pipe; construction methods; serviceability of metal culverts. Before Am. Road Bldrs. Assn.

PORTS AND MARITIME STRUCTURES

DOCKS, SOUTHAMPTON. Southampton Dock Extensions. *Engineer*, vol. 156, nos. 4043, 4044, and 4045, July 7, 1933, pp. 2–5 and 12; July 14, pp. 26–28; and July 21, pp. 52–54, supp. plates. Scheme includes construction of quay wall $1^1/_2$ miles long; dredging of deep-water channel and berths for eight large ships in line; reclamation and raising of over 400 acres of mudland; construction of very large drydock, details of which are given; future extensions.

FRANCE. Les Travaux d'extension by port de Dunkerque, J. Dumas. *Génie Civil*, vol. 103, no. 4, July 22, 1933, pp. 87–90. Progress report on the construction of a new structure at the Port of Dunkirk, France, including concrete piers, about 8 m high.

GREAT BRITAIN. New Works at Southampton Docks. *Concrete and Constr. Eng.*, vol. 28, no. 7, July 1933, pp. 419–425 and 428–432. Description of improvements and extensions at Southampton, England, including a quay wall about $1^1/_2$-miles long, dredging deep water channel, and berths for eight big ships in line; reclamation and raising of about 400 acres of mudland and construction of large graving dock.

HAVANA. Havana—Port of Trade and Travel. *Naut. Gaz.*, vol. 123, no. 11, May 27, 1933, pp. 7–11. Situation of port; fully equipped with modern terminals, warehouses, railway services, and other facilities; complete automatic sprinkler system protects wharves, warehouses, and offices; refrigerating plant.

HAVRE. Port of Havre. *Engineer*, vol. 155, nos. 4039, 4040, 4041, and 4042, June 9, 1933, pp. 566–569; June 16, pp. 592–595; June 23, pp. 616–618; and June 30, pp. 642–645. June 9: Works constructed since 1909; new south pier being constructed progressively from caisson which forms first section of pier-head works, landwards; demolition of old pier; sea embankment. June 15: quays of Bassin de Marée; Joannes-Couvert quay. June 23: Quays of C.I.M.; Quai Nord; oblique mole; drydock in Bassin de Marée.

PIERS, NEW YORK CITY. New York Piers for 1,000-Ft. Ships. *Eng. News-Rec.*, vol. 111, no. 6, Aug. 3, 1933, pp. 123–128. Construction of three piers along the east shore of the North River, 1,100 ft long each, in 15-acre cofferdam 2,039 ft long of steel sheetpile cells; over 1,000,000 cu yd of earth and blasted rock handled in operation; new retaining wall inclosing land portions varies from about 10 to 56 ft in height.

QUAY WALLS, FENDERS. Mechanical Quay Wall Fender. *Shipbldg. and Shipg. Rec.*, vol. 41, no. 25, June 22, 1933, p. 609. Ship-handling equipment installed at port of Le Verdon, France; fenders fitted at intervals of 30 m; each consists of hinged member swinging in a vertical plane at right angles to line of quay wall about axis located at edge of quay; when vessel comes against fenders they are slowly driven inwards; thereafter, fender returns to its normal position, slowly pushing ship with it.

SHORE PROTECTION. Proceedings of Annual Meeting of American Shore and Beach Preservation Association, Washington, D. C., Wednesday, Dec. 14, 1932. *Shore and Beach*, vol. 1, no. 1, April 1933, pp. 1–30. Report includes the following papers: On Keeping Our Coastal Waters Clear for Navigation, J. R. Tiffany; Bill to Provide Federal Aid for Preservation of Public Shores and Beaches, W. H. Sutphin; How Modern Economic Progress Has Multiplied Demand for Outdoor Recreation, L. H. Weir; Need for Beach Attendance Statistics in Order to Promote Cooperative Effort, M. Garsaud; Personal Observations Relating to Value of Coast Protection Works, H. F. Kean; Effects of November Storm on New Jersey Coast, and Degree of Resistance Offered by Various Types of Structures, V. Gelineau; Work of United States Beach Erosion Board, W. J. Barden; and Effect of Recent Storms on Coast Protection Works of Massachusetts, R. K. Hale.

SLIPWAYS, TURNTABLE. 650-Ton Marine Turntable Slipway. *Engineering*, vol. 135, nos. 3515 and 3516, May 26, 1933, pp. 559–561 and 572, supp. plates; and June 9, pp. 629 and 630. Turntable at port of Lorient-Keroman, Brittany, capable of tilting, in addition to revolving, enabling vessels on their cradles to be run off inclined slip on to horizontal radial berths, ten in number, arranged around turntable; slipway is on an incline of 1 in 16; details of haulage gear, cradles, etc.

ROADS AND STREETS

ADMINISTRATION, FINANCING. How the Highway Pays—II and III, R. W. Crum. *Eng. News-Rec.*, vol. 111, nos. 1 and 2, July 6, 1933, pp. 5 and 6; and July 13, pp. 37–39. Vehicle-operating costs as figured for high, intermediate, and low types of roads definitely indicating lower costs on improved surfaces; operating cost of imaginary average automobile in cents per mile; costs of highways balanced against vehicle taxes, savings in operation, and intangible benefits; costs based on liquidation of investment; roadway cost per vehicle mile; résumé of economies; earnings.

AUSTRALIA. Road Construction in Australia, T. J. Cavanagh. *Roads and Road Construction*, vol. 11, no. 126, June 1, 1933, pp. 197 and 198. Activities of various authorities; concrete (cement-penetration types); bituminous emulsions; foundation soils, etc.; heat treatment of clay road; summary of results; timber bridges; road damage by flood; South Australian highway system.

CONCRETE. Curing of Concrete Pavements with Calcium Chloride, H. F. Clemmer. *Roads and Streets*, vol. 76, no. 7, July 1933, pp. 264–267. Favorable curing conditions; moisture supply; vapor pressures; volume changes; relation between silt content of fine aggregate and area of scale; wear and strength results; tests conducted by Iowa and Wisconsin highway departments on volume changes; surface application of calcium chloride.

CONSTRUCTION. Developments in Road Equipment for Asphalt Types of Roads, B. E. Gray. *Eng. and Contract Rec.* (formerly *Contract Rec.*), vol. 47, nos. 25 and 27, June 21, 1933, pp. 607–610; and July 5, pp. 657–659. Review of modern grading machinery, methods of surface treatments, and the preparation of road-mix surfaces and penetration macadam; plant-mixes (both hot and cold); mechanical finishing equipment; reconstruction and salvage.

EXPERIMENTAL. Research Work with Reference to Construction and Repair of Highways, W. P. Robinson. *Surveyor*, vol. 83, no. 2162, June 30, 1933, pp. 687–689. Review of results of 3 years of experimentation on a concrete road at Harmondsworth, Middlesex, England. Before Inst. Mun. and County Engrs.

FOUNTAINS. DRINKING WATER. Simple Cardinal Principles for Roadside Drinking Fountains, J. R. McDermott. *Roads and Streets*, vol. 75, no. 7, July 1933, pp. 259 and 260. Principles of design and examples of roadside drinking fountains in West Virginia.

HIGHWAY ADMINISTRATION. Local Administrative Units for Rural Roads. *Eng. News-Rec.*, vol. 111, no. 4, July 27, 1933, p. 101. Conclusions reached at the conference of the Institute of Public Administration of Social Science Research Council.

HIGHWAY TRAFFIC SIGNS, SIGNALS, AND MARKINGS, GREAT BRITAIN. Traffic Signs. *Surveyor*, vol. 84, no. 2163, July 7, 1933, pp. 9–11. Report of the British Departmental Committee on Road Signs; warning signs, prohibitory signs, mandatory signs, informative signs, misleading signs, carriageway markings, and light signals.

INTERSECTIONS. Basic Design of Roundabout Junctions, H. C. Platts. *Roads and Road Construction*, vol. 11, no. 126, June 1, 1933, pp. 194–196. Conventional lay-out; shape of island; entrances and exits; tangential roundabout; capacity.

MOTOR TRANSPORTATION. Relation of Traffic to Maintenance Costs, B. C. Tiney. *Roads and Road Construction*, vol. 11, no. 126, June 1, 1933, pp. 183 and 184. Study of Michigan State Highway Department of surface maintenance costs per mile for various intensities of traffic; influence of traffic upon condition of concrete pavement; trends of surface maintenance cost under varying traffic intensities. Before 19th Annual Michigan Highway Conference.

RESEARCH. Research Work with Reference to Construction and Repair of Highways, W. P. Robinson. *Inst. Mun. and County Engrs.—Journal*, vol. 60, no. 2, July 18, 1933, pp. 131–148 (discussion) 149–156. Development of highway engineering research in Great Britain, with special reference to the activity of the Road Board of Ministry of Transport; experimental work at Harmondsworth station; testing of experimental road surfaces; details of joints.

ROADSIDE IMPROVEMENT. Road Scenery Unmarred by Signboards. *Contract Rec.*, vol. 47, no. 17, April 26, 1933, pp. 418 and 419. Outline of new Quebec legislation prohibiting posters along roadsides.

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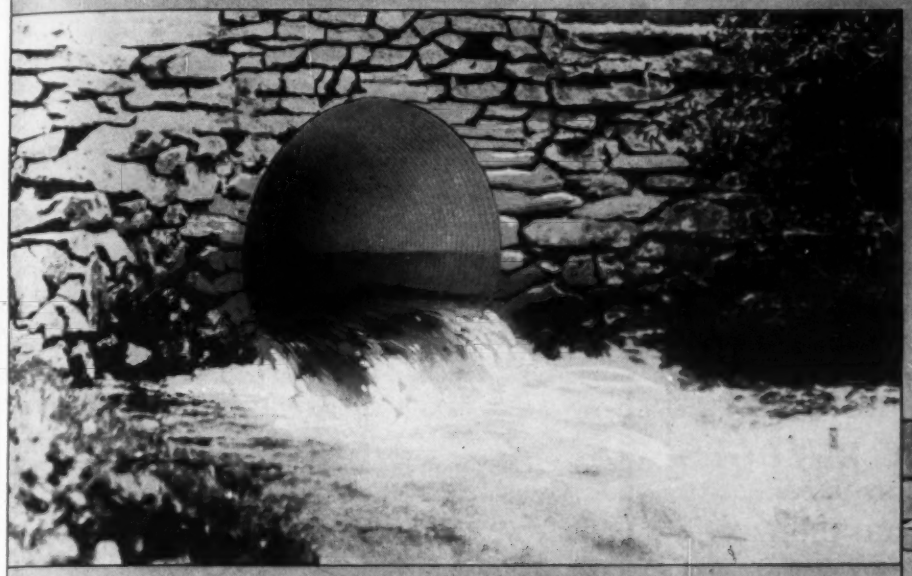

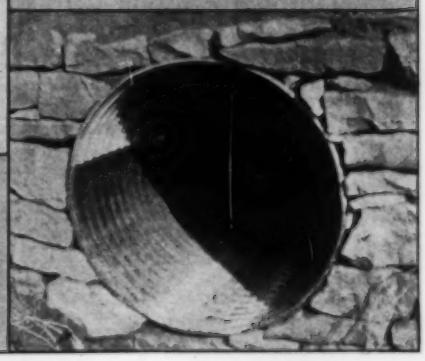

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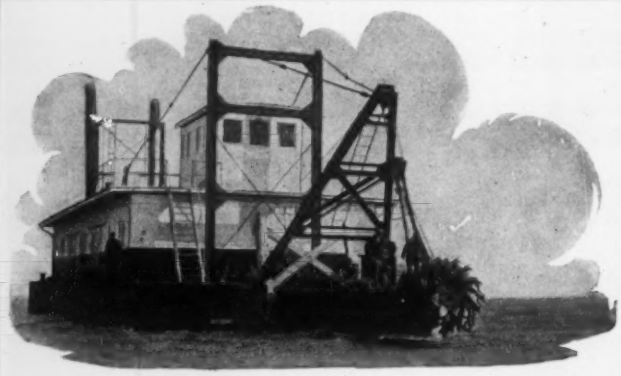

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SEWERAGE AND SEWAGE DISPOSAL

ACTIVATED SLUDGE. Milwaukee Plans Extension to Activated-Sludge Plant. *Eng. News-Rec.*, vol. 111, no. 4, July 27, 1933, pp. 98–101. Critical review of experience of the past 7 years pointing out the necessity of important changes in new works, including two-arm sludge removers to operate in twin tanks, provision for less sludge return, and substitution of circulatory for ridge-and-furrow method of applying air to aeration tanks; proposed extension of Milwaukee sewage treatment works; sludge returns; friction coefficient.

AERATION. Pre-Aeration of Sewage by Air Diffusion, F. C. Roe. *Pub. Works*, vol. 64, no. 7, July 1933, pp. 15 and 16. Use of pre-aeration tanks for grease removal, improvement of sedimentation, and odor control.

CHLORINATION. Diagram for Cost of Sewage Chlorination, C. E. Keefer. *Water Works and Sewerage*, vol. 80, no. 7, July 1933, p. 245. Chart for determining the cost of chlorine treatment of sewage.

COSTS. Costs at Five Sewage Treatment Plants in Illinois, H. E. Hudson, Jr. *Water Works and Sewerage*, vol. 80, no. 7, July 1933, pp. 254–258. Costs of the plants of Bloomington and Normal, Decatur, Elgin, Springfield, and Urbana-Champaign; trend of construction costs; bond interest and retirement; overhead expense; operation and maintenance; annual sewage treatment costs. Bibliography.

FINANCING. Financing Sewerage Works, L. S. Finch. *Am. City*, vol. 48, no. 8, Aug. 1933, pp. 40 and 41. Sewer service charge and revenue bond plan; sanitary district plan; public utility plan.

INDUSTRIAL WASTES. Equitable Adjustment of Charges for Treating Trade Wastes—II, S. T. Powell. *Am. City*, vol. 48, no. 8, Aug. 1933, pp. 44–47. Disposal of industrial wastes by public sewerage systems; estimated operating cost for the treatment of various trade wastes by different biological processes; research and development. Before Soc. Chem. Industry (Concluded.)

INFILTRATION. Infiltration Into Sewers, C. L. H. Humphreys. *Surveyor*, vol. 83, no. 2162, June 30, 1933, pp. 665 and 666. Quantitative determination of infiltration into 7-in. vitrified clay sewers laid in sand fully charged with water, in Berkshire, England.

NEW JERSEY. Proceedings of New Jersey Sewage Works Association, Trenton, N.J., March 23 and 24, 1933, 77 pp. Proceedings of the 1933 annual meeting of the New Jersey Sewage Works Association, including the following papers: Sewer Rentals, C. A. Macdonald, Jr.; Practical Data on Operation of Trickling Filters, I. O. Lacy; Pollution of Bathing Waters and Effect on Health, C. G. Wigley; Chemical Treatment of Sewage in Connection with Sprinkling Filters, J. R. Downes; Brief Sketch of Current Chemical Sewage Treatment, L. L. Hedgepeth.

NEW YORK. Meeting of New York State Sewage Works Association. *Water Works and Sewerage*, vol. 80, no. 7, July 1933, pp. 262–264. Proceeding of 1933 spring meeting, including abstracts of the following papers and discussions: Some Things Learned About Sewerage Aeration by Diffused Air, F. C. Roe; Vacuum Filtration in Sewage Treatment, W. G. Taylor; Problems in Separate Digestion of Sludge, A. Cary; Foaming Troubles, H. Deming; Beautification of Plant Grounds, W. Eckert; Sludge Drying on Covered Beds, A. W. Evans; Operating Routine and Operating Records, J. Nelson; Prechlorination, J. C. Strowbridge; Selection of Sewage Plant Equipment, J. Lewis; Features of Revamped Plant at Newark, N.Y., G. D. Holmes; Operating Experiences at East Rochester, N.Y., R. G. McDonald.

OPERATION. Third-Year Operation Data of Aurora Sewage-Works. *Eng. News-Rec.*, vol. 111, no. 3, July 20, 1933, pp. 67 and 68. Summary of chemical and bacteriological results of sewage treatment; relation between oxygen supply in river and oxygen demand of raw sewage and plant effluent; relation between bacteria, B. coli, dissolved oxygen, and 5-day bio-chemical oxygen demand in Fox River above and below the Aurora sewage-treatment plant.

SEWERS, OUTFALL. Building the Brandywine Outfall Sewer. *Contractors and Engrs. Monthly*, vol. 27, no. 1, July 1933, pp. 18–20. Construction of 6,529 ft of outfall sewer, from 18 in. to 27 in. in diameter, at Wilmington, Del.; unusual conditions faced, with alternate high and low banks, high water trench in narrow dike, and through old rubbish dump; methods of laying pipe; unloading pipe; method of concreting.

STRUCTURAL ENGINEERING

BEAMS, CONTINUOUS. Continuous-Beam Design, R. Fleming. *Eng. News-Rec.*, vol. 111, no. 6, Aug. 10, 1933, p. 176. Discussion of paper entitled, "Continuous Beam Design by Fixed-Point Theory," by O. Albert; previously indexed from issue of June 29, 1933.

DOMES, STEEL. Interesting Steel Dome, *Engineer*, vol. 156, no. 4045, July 21, 1933, p. 69. Illustrations of the dome constructed by Boulton and Paul, which is to be fixed on top of the Roman Catholic church in Bogota, Colombia; structure is 77 ft high to the top of the ornament, with an inside diameter of 31 ft 6 in.; there is approximately 60 tons of steelwork with another 10 tons of ornamental work.

GIRDERS, DESIGN. Theorie und Statik plastischer Traeger des Stahlbaues, K. Eisenmann. *Stahlbau (Supp. to Bautechnik)*, vol. 6, no. 7, March 31, 1933, pp. 54 and 55. Discussion by F. Kann, of paper on the statical theory of plastic steel girders, previously indexed from issue of Feb. 16, 1933; author's reply.

STRESSES, ANALYSIS. Determination of Stresses from Strains on Three Intersecting Gage Lines and Its Application to Actual Tests, W. R. Osgood and R. G. Sturm. *U.S. Bureau of Standards—Journal Research*, vol. 10, no. 5, May 1933, RP559, pp. 685–692, 5 supp. plates. Method is based on dyadic circle, first used by O. Mohr, for determining moments of inertia, later applied by R. Land to plane state of stress, and finally generalized by H. M. Westergaard; applications of method to various projects of Aluminum Research Laboratories.

TUNNELS

VEHICULAR. Driving 4,230-Ft Tunnel in Yosemite National Park, C. S. Moore. *Eng. News-Rec.*, vol. 111, no. 3, July 20, 1933, pp. 76–78. Construction of vehicular tunnel of semi-circular section with a 14-ft radius on a 5-ft springline solving a difficult highway location problem on a major approach route; driving procedure modified as work progressed to increase daily advance from 5.6 to 18 ft; ventilation system; drilling diagrams of tunnel driving; concrete lining and paving; driving, with both heading and bench-drilling; curved bars used in drilling wing holes.

VENTILATION. La ventilation du passage souterrain pour voitures de la Porte Dauphine, à Paris. *Génie Civil*, vol. 103, July 15, 1933, p. 69. Features of ventilating system for vehicular subway, at Port Dauphine, in Paris, having a total length of 250 m. See description of tunnel in *Génie Civil*, June 3, 1933, p. 516.

WATER SUPPLY TUNNELS, CONCRETE LINING. Lining Coast Range Tunnel for Hetch Hetchy Water. *Eng. News-Rec.*, vol. 111, no. 4, July 27, 1933, pp. 107–110. Construction of circular concrete lining for 10.5-ft water-supply tunnel designed to meet wide pressure variations in heavy ground; two-step program for paving invert and pneumatic placing of side walls and arch in three-shift operation; aggregate and batching plant; concreting invert; concreting sides and roof.

WATER PIPE LINES

CROSS CONNECTIONS. When Good Water Becomes Bad, N. N. Wolpert. *Water Works Eng.*, vol. 86, no. 12, June 14, 1933, pp. 586–589. Definition of cross connection; low pressure trouble; presence of typhoid bacilli; epidemics; state instructions; pressure difference leaks; open valve charged with epidemic. Bibliography.

WATER TREATMENT

ACTIVATED CARBON. Evaluation of Activated Carbon, J. R. Baylis. *Water Works and Sewerage*, vol. 80, no. 6, June 1933, pp. 220–224. Developing suitable tests for activated carbon; experimental data available; research in carbon manufacture; comparison of carbons on a basis of their phenol adsorption; procedure for conducting comparison tests on odor removal with carbon.

BACTERIOLOGY. Most Probable Number of B. Coli in Water Analysis, J. K. Hoskins. *Am. Water Works Assn.—Journal*, vol. 25, no. 6, June 1933, pp. 867 and 877. Interpretation of analytical results of water analysis on the basis-of-probability theory. Bibliography.

CHLORINATION. Bacterial Changes in Chlorinated Filter Effluents, H. W. Streeter. *Pub. Works*, vol. 64, no. 7, July 1933, pp. 19–21. Progressive bacterial changes in chlorinated filter effluent; relation between residual chlorine and average bacterial content of chlorinated effluent; effect of residual chlorine; secondary growths of bacteria.

COAGULATION. Water Coagulation with Ferric Chloride, R. N. Clark. *Pub. Works*, vol 64, no. 7, July 1933, p. 22. Tests comparing alum and ferric chloride showing that less of the latter than of alum is needed to effect a given reduction in turbidity; comparison of ferric chloride and alum in coagulation of water; pH range for alum flocculation is less than that for ferric chloride.

CONTROL. Round Table. *Water Works Eng.*, vol. 86, no. 12, June 14, 1933, pp. 596 and 597. Practical discussion of the control of water treatment by pH tests; frequency of pH tests and their use; savings resulting from pH test and other laboratory control.

FILTRATION PLANTS, COSTS. How Much Does Modern Filter Plant Cost? A. D. Stalker. *Eng. and Contract Rec.* (formerly *Contract Rec.*), vol. 47, no. 28, July 12, 1933, pp. 673–675. Analysis of detailed costs of the Ottawa purification works as a guide for estimating the cost of similar undertakings.

LONDON. Treatment of London's Water, A. Houston. *Surveyor*, vol. 83, no. 2158, June 2, 1933, p. 570. Notes on Sir Alexander Houston's Twenty-Seventh Annual Report; maintenance of purity; chlorination; ozone treatment; filtration; bacteriological tests.

ONTARIO. Zeolite Water Softening Plant at Etobicoke Township Water Works, W. H. Walker. *Can. Engr.*, vol. 64, no. 25, June 20, 1933, pp. 9–14 and 19. Description of zeolite down-flow water-softening plant for the treatment of 1.5 mgd derived from deep wells; well construction; removal of iron; operation data; power consumed; savings.

TASTE AND ODOR REMOVAL. Study of Frequency of Tastes and Odors in Philadelphia Water Supply, L. L. Jenne. *Am. City*, vol. 48, no. 7, Aug. 1933, pp. 53 and 54. Locating tastes and odors; filtration equipment; results of taste and odor survey. Before Am. Water Works Assn.

WATER SOFTENING. Present Status of Water Softening. *Pub. Works*, vol. 64, no. 6, June 1933, pp. 17 and 18. Review of recent studies of lime softening and zeolite softening; comparison of processes.

WATER WORKS ENGINEERING

CHICAGO, ILL. Chicago Water Supply System, L. D. Gayton. *Water Works Eng.*, vol. 86, no. 11, May 31, 1933, pp. 475–477. Description of the Chicago water works, including intake cribs, 65 miles of water tunnels, and 12 pumping stations.

DISTRIBUTION SYSTEMS, SERVICE CONNECTIONS. Round Table. *Water Works Eng.*, vol. 86, no. 14, July 12, 1933, pp. 693 and 694. Practical discussion of the installation of services during winter; avoiding hardships of winter field work.

GERMANY. Deutscher Verein von Gas- und Wasserfachmaennern e.V. Berlin W 30, Geisbergstr. 5/6. *Gas- und Wasserfach*, vol. 76, no. 20, May 20, 1933, pp. 341–358. Annual report of the German Society of Gas and Water Engineers, containing a review of the work of research and engineering committees, including standardization of water works.

IRELAND. Limerick Water Supply, F. W. B. Chapman. *Inst. Civil Engrs. Ireland—Trans.*, vol. 59, ninety-seventh session, Nov. 1932 to May 1933, pp. 171–200, (discussion) 201–214, 3 supp. plates. History of the Limerick water works since 1825; description of the present system serving a population of 39,000; water analysis; operation of water filtration plants; cost data.

LAWS AND LEGISLATION. Liability of City or Water Works in Matter of Indebtedness for Improvements, L. T. Parker. *Water Works Eng.*, vol. 86, no. 16, Aug. 9, 1933, pp. 781–783. City entering into contracts where laws do not grant such authority; municipal funding warrants; tax levy exceeding legal limit; assessments; variations in specifications from bond issue.

METERS, MAINTENANCE AND REPAIR. Maintenance of Water Meters—Reading and Billing, J. S. Strohmeyer. *Water Works and Sewerage*, vol. 80, no. 7, July 1933, pp. 267–269. Practice of the Bureau of Water Supply of Baltimore; shop practice; cost of maintenance and repairs; reading practices; meter reader reports; cost of meter reading; complaints. Before Am. Water Works Assn.

RURAL. Pooling of Supplies or "The Grid" in Miniature, T. Marsland. *Engineer*, vol. 156, no. 4046, July 28, 1933, pp. 93 and 94. Summary of author's recommendations; sanitary authority should have control of water supplies in rural districts in preference to small owners; grouping of parishes for purposes of water supply should be promoted wherever possible; hydrogeological survey of area should be undertaken by cooperation between geologist, analyst, medical officer, and engineer. Before Brit. Water Works Assn.

TANKS, MILWAUKEE, WIS. New 6,000,000-Gallon Water Storage Tank, H. H. Brown. *Water Works and Sewerage*, vol. 80, no. 7, July 1933, pp. 237–240. Design and construction of new steel ground-storage tank, solving the distribution problem of Milwaukee, Wis.; this all-welded steel water tank cost approximately $90,000.

WASTE ELIMINATION. Comparison Between English and American Water Waste and Water Consumption, C. M. Saville. *Water Works Eng.*, vol. 86, no. 11, May 31, 1933, pp. 486–490. Conservation of water resources; wholesale metering; wasteful methods in payment for service; rates of American district of 10,000 population; difference between England and America in the use of water; comparison of meter rates.

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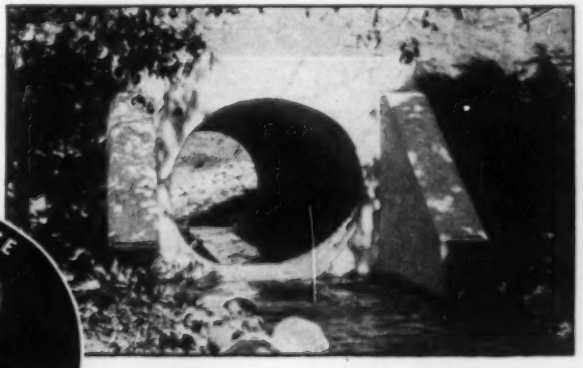

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That's why so many specifications today call for Toncan Copper Molybdenum Iron culverts—culverts made of a modern alloy that hangs on and on in the eternal fight against failure due to rust—that lasts years longer than other ferrous materials, except the stainless steels, that is ready for service when emergency conditions demand safe drainage structures, and that costs less in actual service.

Toncan Iron is available in riveted culvert, plain and perforated, and in sectional plate corrugated pipe up to 15-feet in diameter. In other words, the economical protection of Toncan Iron may now be applied to any conditions—surface drainage, deep drainage, sewers, and small bridge construction.

Either or both of these engineering handbooks will be sent on request—"Toncan Culvert Handbook" —"Toncan Sectional Plate Pipe Handbook." Write today for your copies—they're well worth while.

Gentlemen: Please send complete facts about Toncan Iron Culverts.

*Name*_____

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I am ☐ Engineer ☐ Contractor ☐ Road Official

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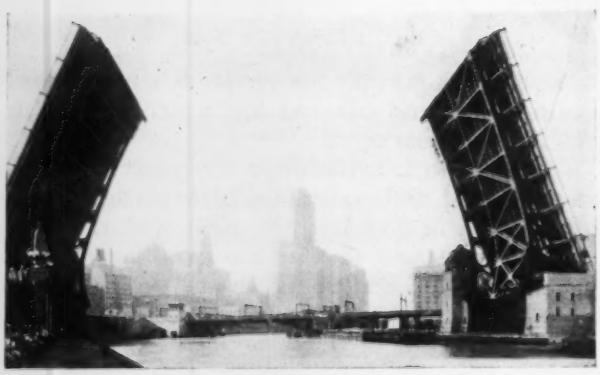

North Wabash Avenue Bascule Bridge, Chicago, Ill.

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